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# GEOTECHNICAL PROPERTIES OF PEACE RIVER GLACIAL LAKE SEDIMENTS

by

© LOK MANYA DATT SHARMA

A THESIS

SUBMITTED TO THE FACULTY OF GRADUATE STUDIES

IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE DEGREE

OF MASTER OF SCIENCE

DEPARTMENT OF CIVIL ENGINEERING

EDMONTON, ALBERTA

SPRING, 1970

Thesis
1970
119

UNIVERSITY OF ALBERTA

FACULTY OF GRADUATE STUDIES

The undersigned certify that they have read, and recommend to the Faculty of Graduate Studies for acceptance, a thesis entitled "Geotechnical Properties of Peace River Glacial Lake Sediments", submitted by LOK MANYA DATT SHARMA in partial fulfilment of the requirements for the degree of Master of Science.

## ABSTRACT

This thesis gives a description of post-glacial lake sediments exposed along a portion of Highway No. 2 near the Town of Peace River in Northern Alberta, and the results of a study of their physical properties which include strength characteristics.

The study was comprised of detailed field work and subsequent laboratory testing.

The deposits are found to be heterogeneous in nature. The laboratory tests reveal that three types of soils have been investigated in this research, namely, laminated lake deposits, with silty and sandy layers; stratified lake deposits, rich in clay sizes; and tills. The deposits have high peak strength but the strength decreases rapidly after peak in the lake sediments. Very little cohesion is found associated with the samples tested. The sensitivity of the deposits is practically unity. Consolidation test results are complex and proper assessment of the geologic history is difficult from the test data. The detailed geological sections suggest a complex Pleistocene history for the strata. Laminated lake sediments are found overlain by sand, tils and silts.

Recommendations for further research are made to study consolidation characteristics, residual strength, ground water conditions and the Pleistocene Geology of the area.

# ACKNOWLEDGEMENTS

The author wishes to acknowledge his indebtedness to the following for their assistance in the preparation of this thesis.

Dr. S. Thomson, Professor of Civil Engineering for his guidance and encouragement during the course of this investigation.

Dr. N.R. Morgenstern, Professor of Civil Engineering, for his patience, perseverence, friendly advice and useful discussions concerning this work.

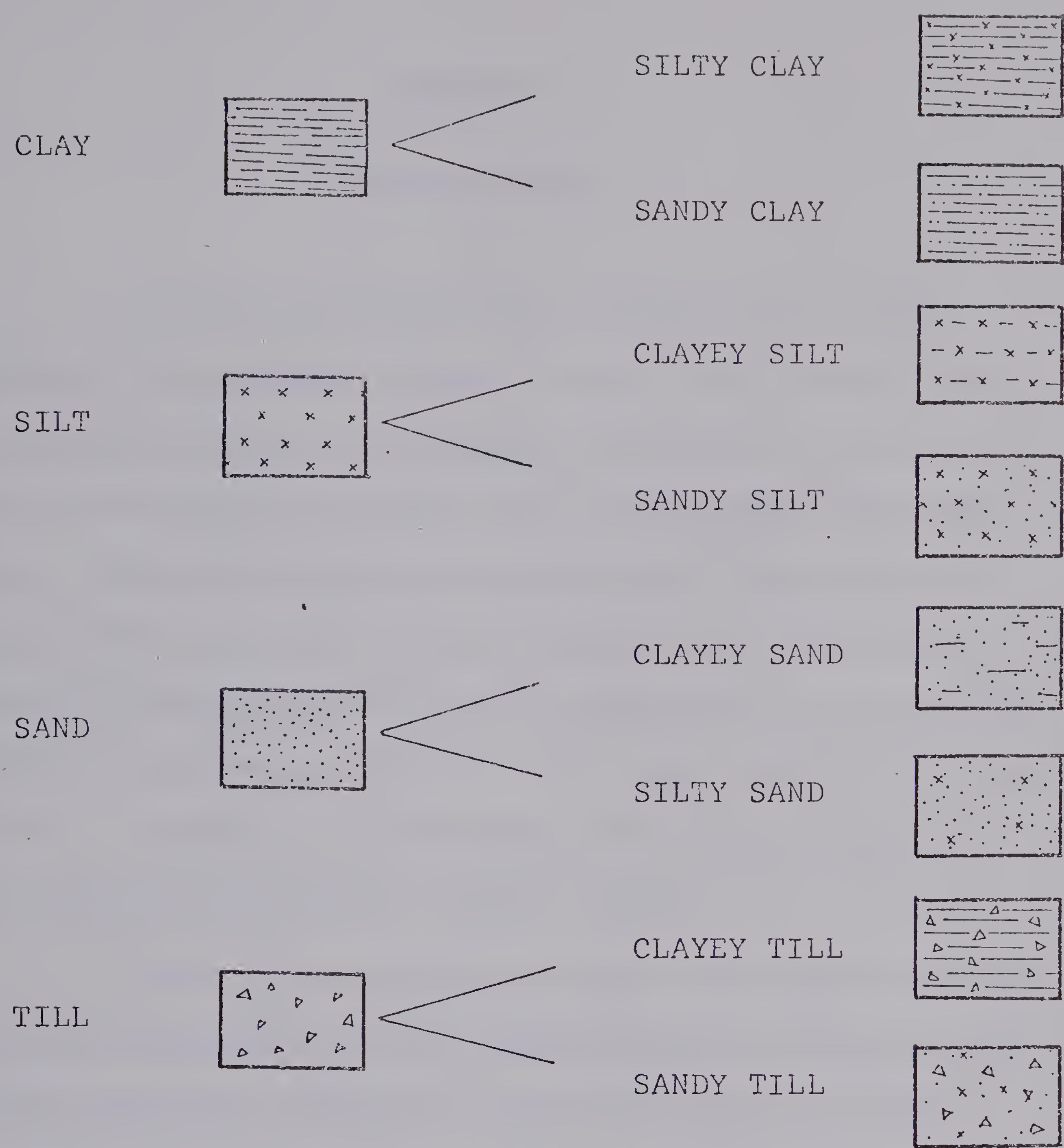
The National Research Council of Canada for providing funds for the project.

TABLE OF CONTENTS

LIST OF TABLES

## LIST OF FIGURES

## LIST OF PLATES




SYMBOLS FOR SOIL TYPES USED IN THE THESIS

# CHAPTER I

## INTRODUCTION

The Alberta Department of Highways recently completed a new access highway to the Town of Peace River in Northern Alberta. The highway traverses an area of old landslides along the valley walls of the Heart and Peace Rivers. The work required deep cuts and high fills at places which upset the natural stability of the slopes. Preliminary investigations of soil conditions and location of possible alignments started in 1964, and construction began in 1965. A number of landslides were encountered in the area during construction (Pennel, 1969)*.

During the construction work, the deep cuttings along the alignment of the highway exposed interesting sub-surface deposits, comprising glacial tills and glacio-lacustrine deposits. The geotechnical properties of these deposits had not previously been studied.

The research comprising this thesis was a study of the behaviour of these sub-surface deposits along a portion of the highway (Highway No. 2). The thesis gives a description of the post-glacial lake sediments of the area and the results of a study of their physical properties which

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* List of references is given at the end of this thesis.

includes strength characteristics. The study is based on detailed field mapping of three typical sections as revealed in the backslopes and subsequent laboratory testing of block samples, shelby tube samples and bag samples.

Pennell (1969) carried out residual strength tests and analysis of slopes which were largely composed of glacial deposits whereas the present study has been carried out mainly on glacial-lake sediments in the area.

Geotechnical studies of glacial-lake clays have been reported by Wu (1958), Milligan *et al* (1962), Soderman *et al* (1960) and their work indicates that glacial lake deposits generally exhibit low strength and high compressibility.

The scope of the present research is limited to the sites shown in Figure IV-1 in Chapter IV. Geographical location of the sites at Peace River is shown in Figure I-1. The geology of specific sites was studied in detail. Block samples, shelby tube samples and bag samples were collected. The backslopes in three typical study areas were mapped in detail. Included in this thesis are some supplementary data from the work of the Alberta Department of Highways.

Detailed piezometric studies were beyond the scope of this thesis. No attempt was made to analyze any of the existing slopes because of lack of information regarding the pore water pressures. In addition the study area is heterogeneous.

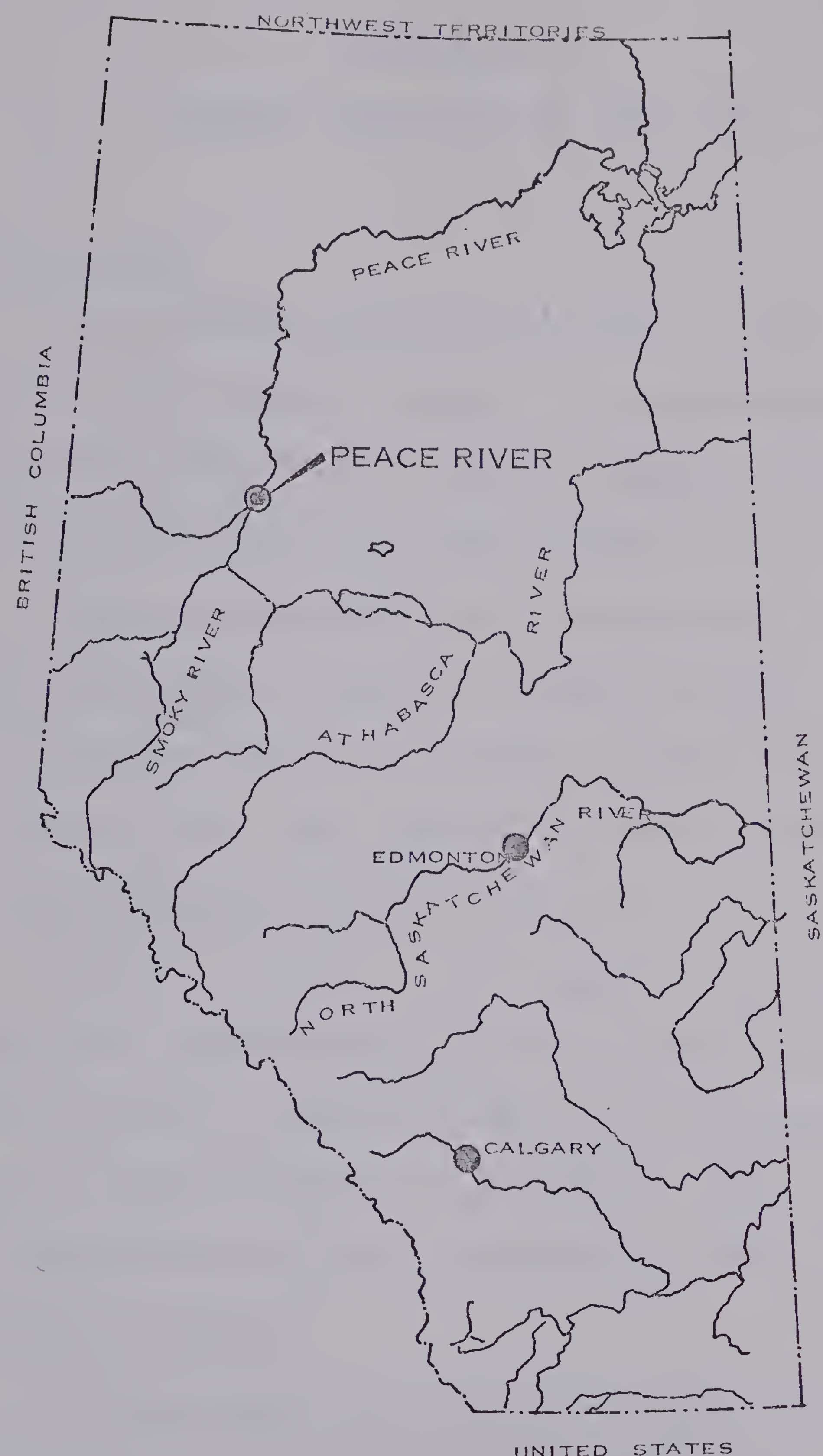



FIGURE I-I. PLAN SHOWING LOCATION OF SITE

# CHAPTER II

# GENERAL CHARACTER OF THE AREA

## 2.1 Physiography

The general surface topography in the area is composed of three elements namely - the semi-continuous uplands, broad prairies with gentle slopes, and deeply incised river valleys with steep slopes.

The maximum difference in elevation between the Peace River bed and the upland is about 2000 ft. The deep valleys of streams and the uplands, in fact are the distinct features that divide the area into various districts with respect to population distribution.

Present features of the general study area are the results of erosion and deposition over a long period. Three main stages in the development of the present surface are considered to have occurred (Rutherford, 1930) - preglacial erosion; glacial erosion and deposition; and post-glacial erosion and deposition.

It is probable that prior to glaciation, the slopes from the river level to the uplands were more regular and gentle than at present. According to Rutherford (1930) two causes have contributed to this change of surface - filling

of the depressions by glacial and glacial-lake deposits and a subsequent development of deeper river valleys. The glacio-lacustrine deposits in the area are exposed in steep cliffs along the river valleys and in the cuts made along Highway No. 2 near the Town of Peace River. These deposits disintegrate readily and are one of the contributing causes to slumping along the valley walls.

It is difficult to ascertain the thickness of the glacial-lake deposits. Thicker exposures occur along the main river valleys and thin out in the direction of the uplands. The fine texture and argillaceous composition of these deposits renders them impermeable to ground water. Localized sand and silt pockets are also prevalent.

The Peace River and its tributary, the Smoky River, are the two main streams in the Peace River District at present. The other rivers and creeks are tributaries to these two main streams. The Heart River joins the Peace River at Peace River town. The Heart River is a permanent stream about 50 ft wide and 6 to 12 inches deep during the summer.

## 2.2 Geology

The following abstract of the geology of the area is from the works of Rutherford (1930), Crockford (1949), Jones (1966), Tokarsky (1967), Henderson (1959), and Mathews (1963).

### 2.2.1 Bedrock Geology

Bedrock strata underlying the general area are of Cretaceous age and consist of series of sandstones, and marine shales. The strata in general dip to the southwest and thicken in the same direction. The regional slope of the land is to the north. Thus the lower formations crop out to the north and younger to the south. Geological structure is relatively simple except for some minor folds of local importance. Various bedrock formations are exposed chiefly along the valleys of the major streams. Most of the Cretaceous strata are quite soft and tend to slump readily (Jones 1966).

The lower strata exposed in the area are sandstones and shales of the Peace River Formation, which outcrop along the Peace River from Peace River Town northwards. Crockford (1949) suggests the source of the sediments of the Peace River Formation to be west of the present Rocky Mountains.

Overlying the Peace River Formation are the Shaftesbury shales, consisting of marine shales and silty shales, which outcrop along the Smoky and Peace Rivers. This formation thins out eastwards and southward. The uppermost part of the formation becomes silty and sandy and is transitional into the overlying Dunvegan sandstones. The Dunvegan Formation, which crops out prominently on the north side of the Peace River at the site of Fort Dunvegan, consists of a series of sandstones and shales.

Upper Cretaceous marine shales overlying the Dunvegan Formation have been referred to as the Smoky River Group by Dawson (1879), which according to McLearn (1918) consists of three members, namely - Upper Shales, Badheart Sandstones, and Lower Shales. Table II.1 shows the stratigraphic relationship of the various formations of the general area.

### 2.2.2 Buried Channel Deposits

The term "buried channel deposits" has been used in the geologic literature for deposits of sand, gravel, silt and clays overlying the Cretaceous bedrock and underlying the glacial surfical deposits (see Table II.1). Buried channels in the Peace River area have been recognized by many authors. Rutherford (1930) mentioned thin layers of gravel resting on bedrock exposed along the rivers in the Peace River country. Jones (1966) suggested that most of the buried sand and gravel deposits were the probable northern equivalent of the Saskatchewan sand and gravel deposits described by Rutherford (1937). Jones (1966) favoured a pre-glacial origin for these deposits. Mathews (1963) reported similar deposits exposed along the banks of Peace River and its tributaries in Fort St. John area in Northern British Columbia directly west of the present study area. He considered these deposits to be early interglacial or pre-glacial in age.

TABLE II.1

TABLE OF FORMATIONS (EXPANDED FROM GREEN AND MELLON, 1962)

| Era | Period or Epoch | Rock Unit | | | Thickness (feet) | Lithology Lithology |
|---|---|---|---|---|---|---|
| Cenozoic | Recent | | | | 0-100+ | River alluvium and terrace sand, silt, gravel; lacustrine and flood-plain sand, silt, clay; muskeg; eroded slopes, slump and slide debris. |
| | Pleistocene | | | | 0-650 | Till; glaciofluvial sand & clay; glaciolacustrine clay, silt, sand, gravel. |
| | Tertiary | Buried Channel Deposits | Deeply buried | | 0-350+ | Gravel, sand, silt, clay. Graimshaw gravels and the intermediate level deposits form important acquifers in Grimshaw area. |
| | | | Intermediate level | | 0-100+ | |
| | | | Grimshaw gravels | | 0-130 | |
| Mesozoic | Upper Cretaceous | Smoky River Group | Puskwaskau Formation | | 0-130 | Shale, dark grey, fissile |
| | | | Bad Heart Equivalent | | 0- 10 | Sandy shale, greenish; siliceous shale & black rusty sandstone. |
| | | | Kaskapau Formation | Upper Member | 0-230 | Shale, dark grey, fissile |
| | | | | Lower Member | 0-140 | Sandy & silty shale; whitish sandstone |
| | | Dunvegan Formation | | | 0-405 | Sandstone, soft, grey; grey, silty carbonaceous shale; the only important bedrock aquifer in Cardinal Lake-Grimshaw area |
| | | Shaftesbury Formation | | Upper Member | 0-460 | Silty shale, grey; thin laminated siltstone |
| | | | | Lower Member | 270-460 | Shale, black fissile; fish scales horizon near top |
| | Lower Cretaceous | Peace River Formation | | Paddy Member | 40-80 | Sandstone, poorly sorted carbonaceous; one or more thin shaly or coaly zones |
| | | | | Cadotte Member | 60-85 | Sandstone, massive, fine grained to very fine grained sandstone |
| | | | | Harmon Member | 40-70 | Shale, dark grey; thin lenses of siltstone & very fine grained sandstone |

(After Tokarsky, 1967)

Outcrops of the deeply buried channels are scarce in the study area because of the widespread large scale slumping along the river valleys. The slumping in the Heart River valley is extensive probably due to the fact that the valley has been cut in a sequence of thick surficial deposits. The base of the gravel in these buried deposits is at approximate elevation 1150 ft. (Tokarsky, 1967).

Buried channel deposits consist mainly of fine pebble-sized, well sorted, rounded gravel with interbedded sand (Tokarsky, 1967). These gravels are mostly quartzites of Rocky Mountains origin but contain a few gneiss and granite pebbles typical of the Canadian Shield. These gravels are overlain by a resistant unit of hard platy clay about 24 ft. thick. Figure II.1 shows an outcrop section near the town of Peace River.

### 2.2.3 Surficial Deposits

The surficial deposits consist of till, glacio-lacustrine sands, clays and silts, and glacio-fluvial sands and gravels.

Tills of the Peace River area are very clayey and underlie the glacio-lacustrine deposits. Two types of tills are recognized in the area (Tokarsky, 1967). These are a northern till containing rocks of Canadian Shield origin and locally derived rocks from the underlying bedrock, and a southern till in which locally derived rounded pebbles predominate. These pebbles are from the Grimshaw gravels which

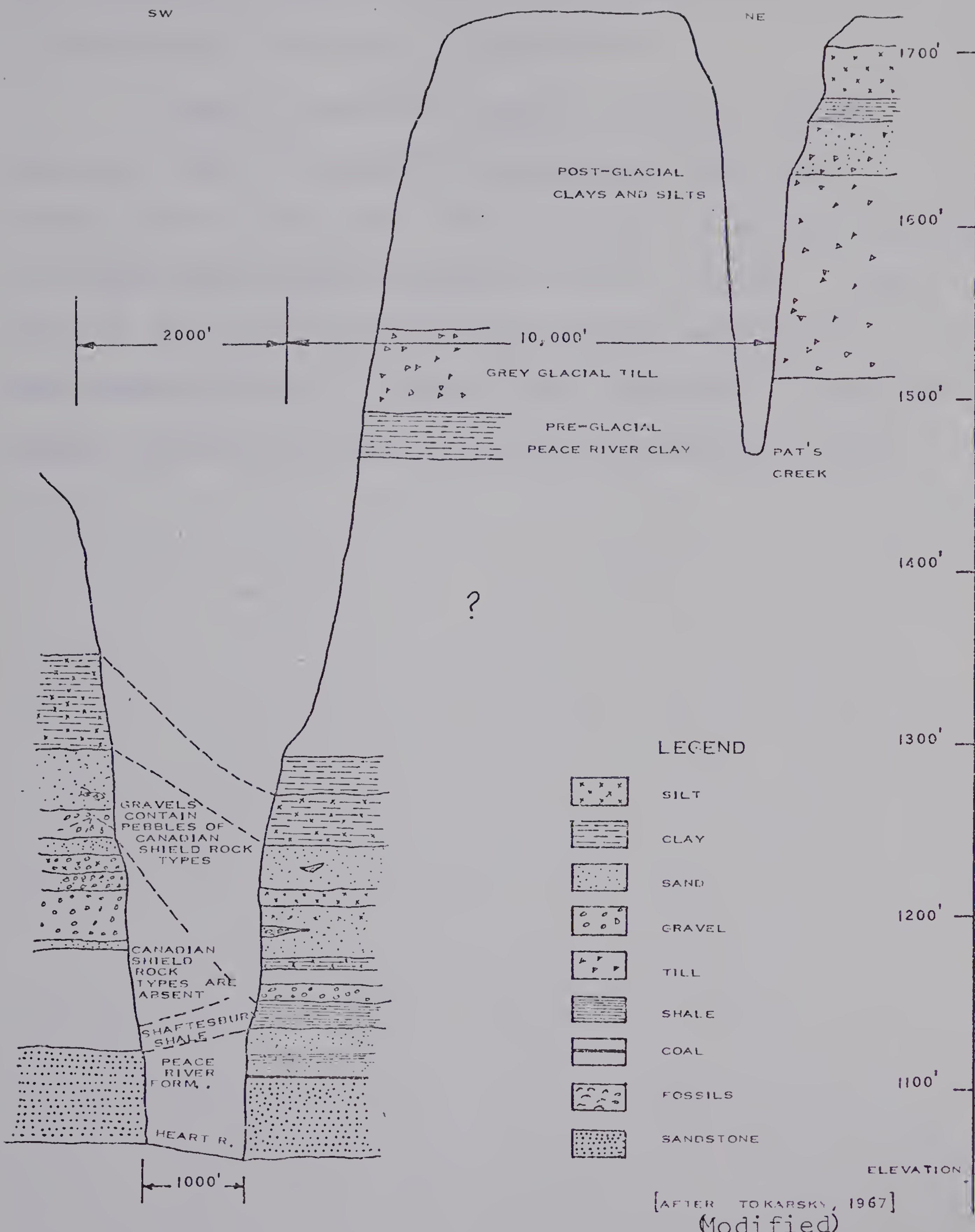



FIGURE II.1. OUTCROP SECTION NEAR PEACE RIVER TOWN

are, at present, being mined in the Grimshaw area some 15 miles west of the town of Peace River.

Glacio-lacustrine deposits consist of sands, silts and clays, laid down in proglacial lakes that covered much of the area. Most of these lacustrine deposits are bedded and perhaps should be termed "varved". These deposits form the parent material of the soils in the Peace River District. Glacial beach deposits of sand and gravels are also located in the area (Henderson, 1959).

# CHAPTER III

# REVIEW OF LITERATURE ON GLACIAL LAKE CLAYS

## 3.1 Notes on Glacial Lake Deposits

Many glacial lakes were formed during the Pleistocene Epoch in North America. Soft, fresh-water clay sediments have been typical of these glacial lakes. A unique combination of geology, topography and climatic conditions are usually found accompanying the advance and retreat of glacial ice (Walker and Irwin, 1954). The rhythmically banded sediments typical of a glacial lake have been referred to as varved clay by many authors. However, the geology and geochronology of varved clays is open to dispute. G. DeGeer in proposing the term varve, applied it to both marine and fresh-water deposits (Milligan *et al.*, 1962).

The mode of formation of varved clays is not clearly understood. Various contradicting theories have been postulated for the conditions for the formation of varved clays (Antevs, 1930; R.E. Deane, 1950; Legget and Bartley, 1953). Antevs' (1930) hypothesis of annual deposition has been questioned by R.E. Deane (1950). Milligan *et al.* (1962) have referred to the two distinct layers of a varve as being called by various names such as silt and clay layers,

light and dark layers or summer and winter layers. The sequence in many varved clays may, however, involve three layers of various colours (Deane, R.E., 1960).

The layers of a banded sediments reflect the periodic changes in the character of the material that enters the region of sedimentation. The geologic processes in the origin of laminated clays may be so complex that there is always a possibility of large variation in the thickness of layers, and within the deposits, of extreme variation in their properties.

The geographic location of the major glacial lakes of Canada is given in the Glacial Map of Canada produced by the Geological Survey of Canada. Laminated sediments are not found everywhere within these lake areas. They may be found in isolated pockets remote from the boundaries of the now extinct glacial lakes, probably caused by temporary pondings between glacial stages (Deane, R.E., 1960).

Laminated clays present an unusual problem for normal soil mechanics analysis. The normal analysis based on homogeneous soils may be found invalid for such deposits. Testing methods can overestimate the in-situ strength of banded sediments, which may in part be due to sample disturbance. Since there are many possible modes of deposition, the individual layers may differ in structure. The results

of tests on bulk sample can be misleading and considerable caution must be applied in generalizing the geotechnical properties of such deposits.

## 3.2 Notes on Basic Shear Strength Properties

In the following section, after some general notes on types of clays, a brief survey is made of the basic shear strength properties. The discussion is diverted more towards the behaviour of clays.

Terzaghi (1936) put forward three-fold classification for clays, namely, soft, stiff intact, and stiff fissured. He distinguished between stiff and soft clays on the basis of liquidity index suggesting that soft clays have liquidity indices equal to or greater than 0.5, whereas stiff clays have liquidity indices typically close to zero. For sedimentary clays Terzaghi stated that soft clays are normally or lightly overconsolidated, whereas, the stiff clays are heavily overconsolidated. Skempton and Hutchinson (1969) modified this distinction between clays by adding that the cohesion intercept, expressed in terms of effective stress, is zero or very small in soft clays, whereas, the intercept for stiff clays is appreciable. It should, however, be pointed out that both the cohesion intercept, and liquidity index of less than 0.5 may be found associated with recent deposits having been overconsolidated by desiccation.

Terzaghi's term intact clay means free from joints and fissures. Fissured clays contain a network of structural discontinuities. The term "macrostructure" has been used by Morgenstern and Tchalenko (1967) for those features of clay fabric which can be seen by the unaided eye. Fissures, joints, slickensides and beddings are the macrostructures usually recognized in clays. Desiccation causes macrostructures in recent deposits. Rosenqvist (1955) suggested that phenomenon of fissures seems to be related in some way to the electrolyte content of the clays and fissures may even be found in soils where no drying has occurred and where the water content is still much above the shrinkage limit.

Skempton (1969) further subdivided the groups of clays on the basis of liquid limit. He presented the following suggestions:

| Clay Type | Liquid Limit |
|---|---|
| Sandy or silty clays | $\angle$ 30 |
| Clays of low plasticity | 30 - 50 |
| Clays of medium plasticity | 50 - 90 |
| Clays of high plasticity | $>$ 90 |

Clays may also be classified according to their origin and mode of formation such as sedimentary clays, glacial clays, peri-glacial clays, clays transported by landslides, and clays produced by in-situ rock weathering.

### 3.2.1 Anisotropy

Anisotropy is normally expected in clays as a consequence of their mode of formation. Effects of anisotropy on shear strength may be appreciable. The strength parameters $c'$ and $\phi'$ are likely to be dependent to some extent on orientation of the specimen during testing. If a pre-existing slip surface is present in a deposit, the strength along this surface is at or close to the residual angle of shearing resistance and therefore, may be lower than the strength in any direction. Duncan and Seed (1966) and Ladd (1966) give an account of effects of anisotropy in clays.

### 3.2.2 Mohr-Coulomb Failure Criteria

Mohr-Coulomb theory, in terms of effective stresses, has been generally accepted in stress analysis and design. In the following discussion its validity will be examined.

The theory assumes that strength is dependent only on the major and minor principal stresses. The theory is supported by the results of triaxial tests. In triaxial test the intermediate principal stress, $\sigma_2$, and the minor principal stress, $\sigma_3$, are the same (cell pressure), whereas, in most field problems $\sigma_2$ and $\sigma_3$ are not equal.

Investigations made by Habib (1953), Bjerrum and Kummeneje (1961), and Cornforth (1964) show that strength deviates somewhat from that predicted by Mohr-Coulomb theory if $\sigma_2$ is different from either $\sigma_1$ or $\sigma_3$.

Direct field evidence of this would, in fact, be the shear strength exhibited by soil masses during slides and by foundation failures. Slopes investigated by Skempton (1948) and Ireland (1954) as reported by Wu (1968) show that the discrepancy between the calculated and actual factors of safety is less than 15 percent.

For normally consolidated clays, Skempton and Bishop (1954) gave the following relation between consolidation pressure and shear strength:

$$\frac{C_u}{P_o} = \frac{\text{Sin } \phi' \left[K_o + A(1 + K_o)\right]}{1 + (2A - 1) \text{ Sin } \phi'}$$

where,

$C_u$ = Undrained strength

$P_o$ = Effective overburden pressure

$K_o$ = Ratio of horizontal to vertical effective stresses

$A$ = Skempton's pore pressure parameter

$\phi'$ = Effective friction angle

This equation shows that $C_u$ is proportional to $P_o$ and the proportionality depends on the properties $\phi'$, A and $K_o$. Bjerrum and Simons (1960) show that the values of $C_u/P_o$ calculated from laboratory determined $\phi'$, A, and $K_o$ are in satisfactory agreement with those measured in-situ, which supports the Mohr-Coulomb criterion.

This criterion is in better accordance with experimental results than any of the more elaborate formulations at present available (Bishop, 1966).

### 3.2.3 Peak and Residual Strength

Peak strength of clays may be defined simply as the peak point of the stress-strain curve. Stress-strain curves for clays drop off following the peak and this drop off may be quite pronounced in natural soils. The ultimate value of the stress at large strains has been referred to as the residual strength, ultimate strength, or even critical state by various authors.

Skempton (1964) applied the concept of residual strength to the systematic tendency for strength of stiff fissured clays in cuttings to diminish with time. It also explained the fact that old landslides are sensitive to disturbances.

Kenney (1967) suggested that the residual strength is primarily dependent on the mineralogical composition. Although progressive breaking of adhesive bonds may play a role in decreasing the strength in normally consolidated clays, a second factor that would also seem to be important is the gradual reorientation of clay particles into parallel, face to face arrangement. Increase in water content due to dilatancy within the zone of shear also aids in decreasing the strength at large strains (Skempton and Hutchinson, 1969).

Skempton and Petley (1967) show that the tests done on samples with a pre-cut shear plane exhibit a reasonably good agreement with the strength measured in reversing

shear box tests on an initially unsheared sample. However, Skempton and Hutchinson (1969) report that recent tests using a ring shear apparatus, indicate that before a strictly constant ultimate strength is reached very large displacements (of the order of one meter) are necessary, on a perfectly plane surface, and this ultimate residual may be much lower than the strength in reversing or pre-cut plane tests.

3.2.4 Drained and Undrained Strength

The shear strength of clays may be expressed in two forms. The first is to use the effective stress envelope obtained from drained tests or consolidated undrained test with pore pressure measurements. The effective stress parameters $c'$ and $\phi'$ allow the determination of the shear strength at a point in a soil mass if the effective normal stress at that point is known.

For saturated soils the second method is to express shear strength in terms of undrained strength in terms off. total stress. The undrained strength is determined by tests in which no overall water content change is allowed to occur during application of the shear stress. The undrained test measures the strength of the soil before construction. Determination of appropriate undrained strength in clays becomes difficult in presence of micro and macrostructures. The undrained strength depends on the factors like water content and stress history (Skempton and Sowa, 1963). Also the undrained strength of a soil depends upon the rate at

which volume change takes place in the soil. In clays, the rate of volume change is so low (because of low permeability) that the shear strength may accurately be represented by undrained strength. However, the evaluation of the shear strength at a given time requires determination of the magnitude of the pore pressures in the field.

In presence of micro and macrostructures, the representative undrained strength may be obtained by large scale testing. However, Chandler (1968) reports that overestimation of strength may be recorded during undrained triaxial compression tests when failure results in the formation of one or more shear planes.

The effects of anisotropy on undrained strength may be appreciable. The parameters $c'$ and $\phi'$ are influenced by specimen orientation, also the pore pressure response to a given stress change varies with direction as a result of $K_o$ consolidation conditions (Hansen and Gibson, 1949). Balsubramonian and Eigenbrod (1969) gave a theoretical approach to the influence of anisotropy and stress reorientation on undrained strength.

## 3.3 Geotechnical Mapping

Failure to appraise correctly the geological and geotechnical data concerning surface and sub-surface strata sometimes necessitates distinct changes in original engineer-

ing designs. The conventional geological maps available lack the detail needed for design and hence may not exert sufficient influence on the original designs. Therefore, a map with more emphasis on representing engineering properties of the substrata is needed. In the literature this type of map has been referred to as geotechnical map. In the following section a brief outline of geotechnical mapping is presented (Fookes, 1969).

The general purpose geological maps often ignore the superficial deposits overlying the bedrock and give greater emphasis to the details of structural and stratigraphical information in the rock within the area. A geotechnical map is an improvement of a geological map for engineering purposes. The ultimate aim of geotechnical mapping is to produce a map whose units are defined by engineering properties or behaviour and unit boundaries shown in terms of change in property. However, a difficulty encountered in this technique would be due to the fact that the physical properties of rock and soils are usually gradational and no definite line could be drawn between various units.

Fookes (1969) has presented methods of geotechnical mapping of soils and sedimentary rocks with practical example of practice from the Mangla Dam project. He classified geotechnical mapping into four units:

1. Mapping in terms of descriptive soil or rock classifications.

2. Mapping in terms of specific engineering property.

3. Mapping in terms of a limited number of properties.

4. Mapping in terms of general geology plus additional engineering informations and inference.

A. Peter (1966), Fookes (1969) have developed various techniques of graphical representations, which help in visualizing the nature of the subsurface.

CHAPTER IV

# SITE INVESTIGATION AND SAMPLING

## 4.1 The Site

The geographical location of the site of the field investigations is shown in Figure I-1, and the site plan is shown in Figure IV-1. The site is subdivided into three areas - Project Area A, Project Area B, and Project Area C. The alignment and chainage of Highway No. 2, the position of borings of the Alberta Department of Highways and the Project Areas are also shown on Figure IV-1. A preliminary study of the area was made with the help of the borehole data available and air photographs of the area.

## 4.2 Detailed Exploration

Detailed explorations were carried out to obtain the soil profiles and undisturbed samples. The cut slopes exposed along the highway were used for the exploration and mapping.

### 4.2.1 Geotechnical Mapping

Field mapping was done for the cut slopes to obtain descriptive information concerning the soil strata.

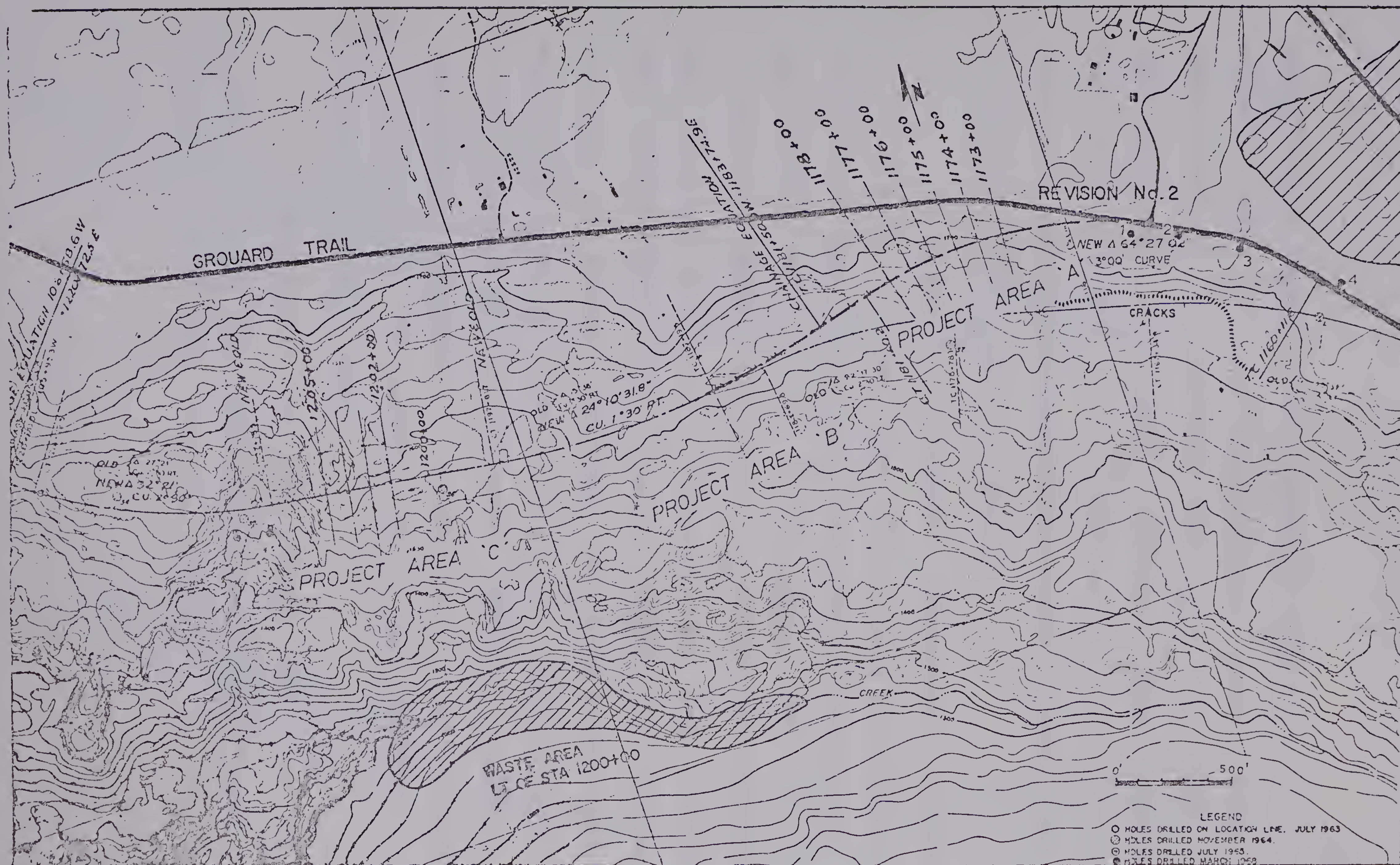
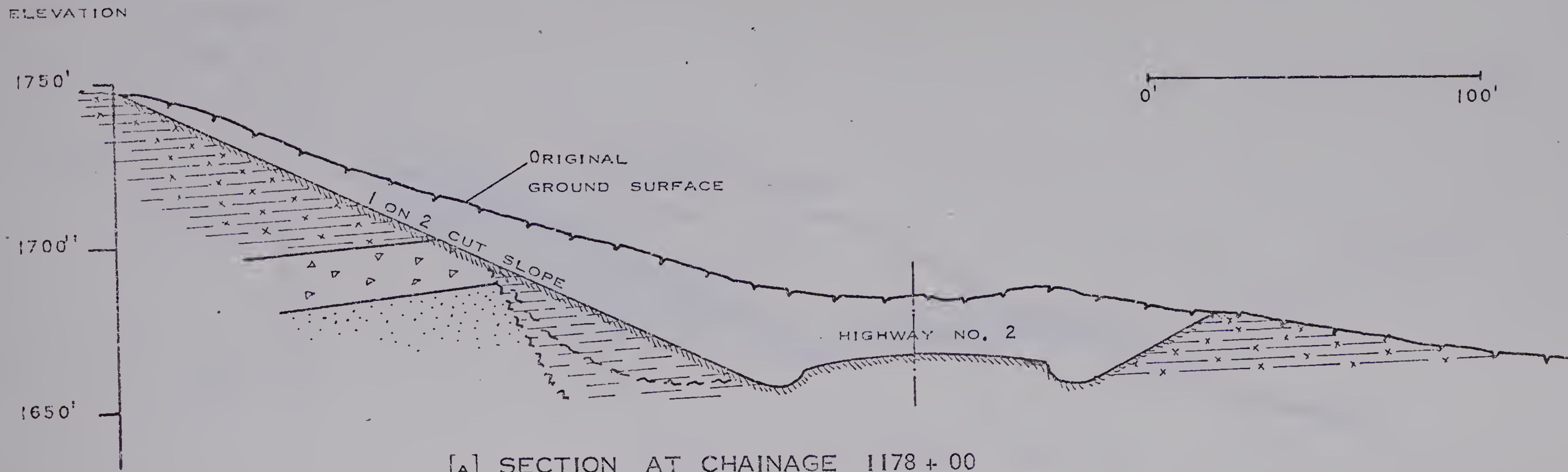



FIGURE IV-1, SITE PLAN

Deposits were logged to show bedding, weathering, colour, estimated grain size and consistency, and local cracks. Data in Appendix A gives the description thus obtained and Figures IV-2, IV-3, and IV-4 show the logged sections. The various positions of the logged sections are shown in Figure IV-1 and are referred to the chainage points used by the Alberta Department of Highways.

The procedures followed in logging the sections was as follows. A survey tape 100 feet long was fixed to a peg at the top of the slopes and extended along a line approximately perpendicular to the highway. The distances of various boundaries of changes in soil type were measured down and along the slopes. A hand pick was used in digging the soil for the visual study. A Brunton compass was utilized to measure the dips and bearings. The slope angles were measured by means of an Abney Hand Level. Observations of the soil type were made visually, however, a pocket penetrometer was employed to estimate the in-situ strength of the materials. The sections thus logged were then correlated with each other.

Differential levelling was done along the foot of the slopes and the slope distances were tied to the elevations at the foot of the cut slopes.

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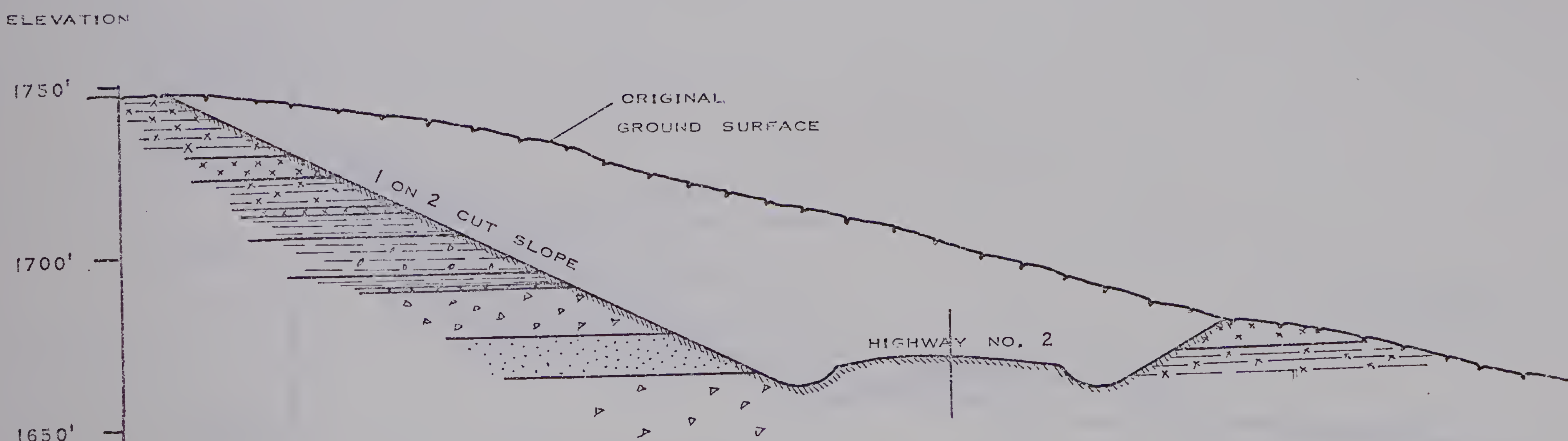
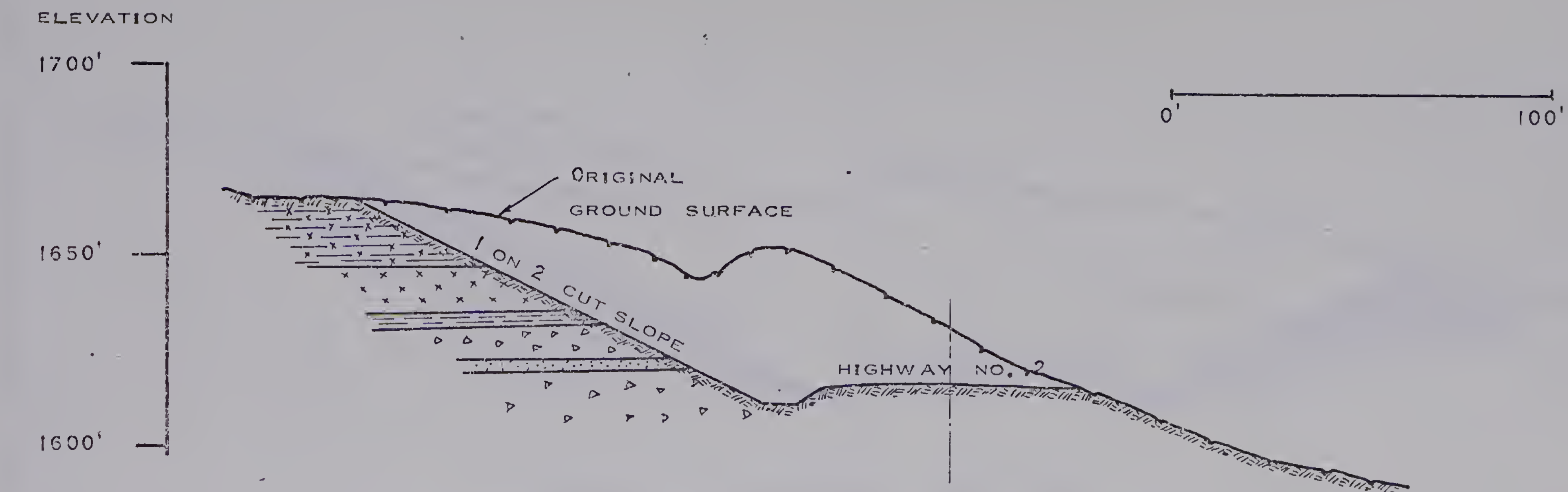



[B] SECTION AT CHAINAGE 1176 + 00

FIGURE IV-2 -PROJECT AREA 'A' - SECTIONS ALONG HIGHWAY NO. 2, PEACE RIVER TOWN.

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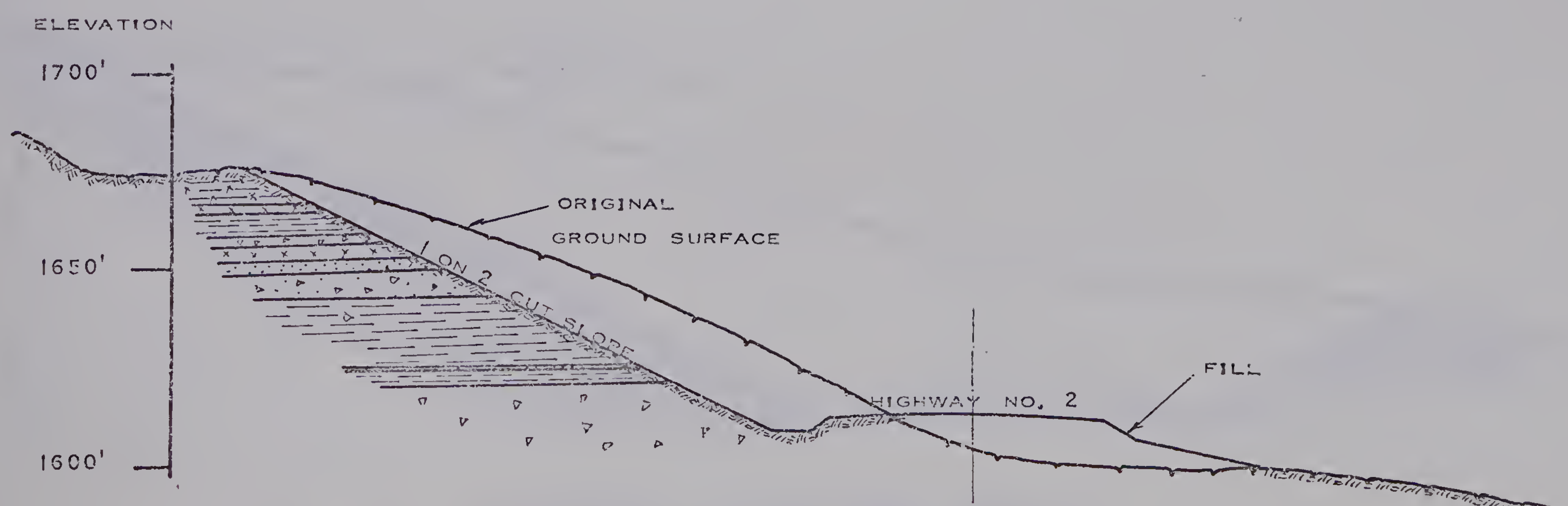
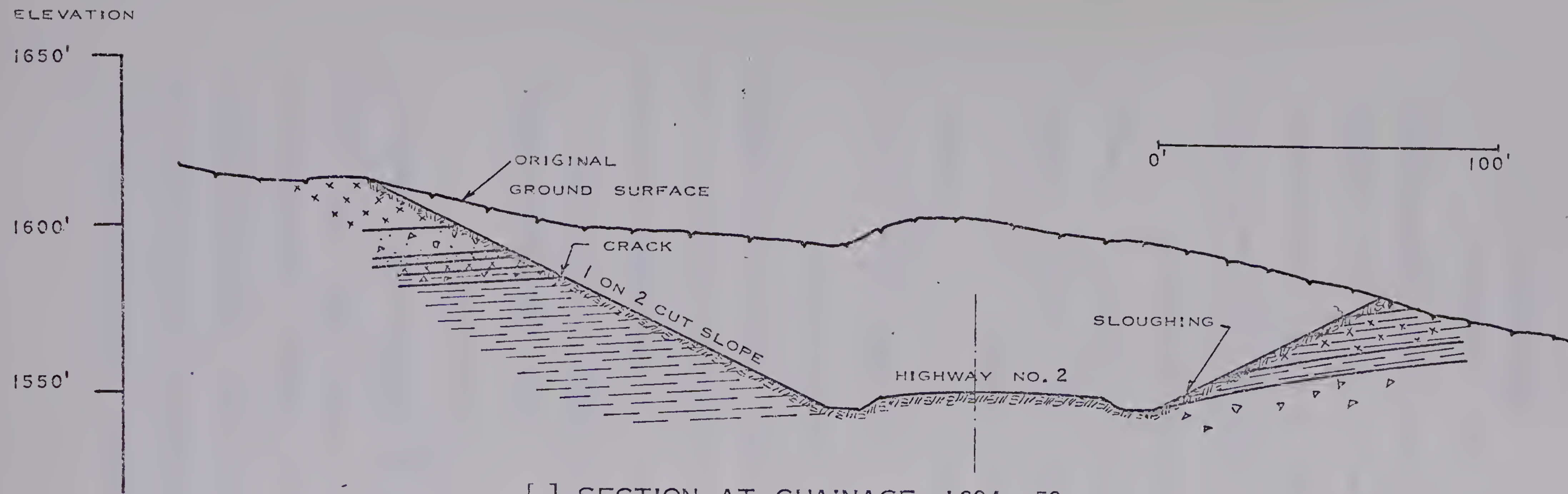
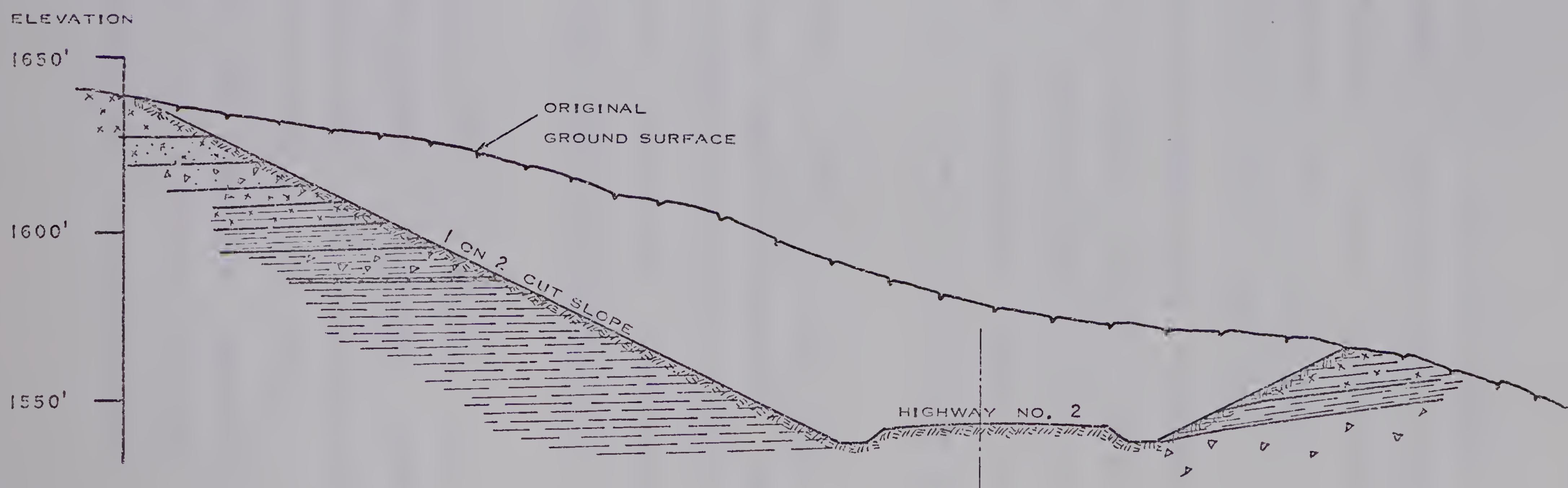



[B] SECTION AT CHAINAGE 1190 + 00

FIGURE IV-3. -PROJECT AREA 'B' – SECTIONS ALONG HIGHWAY NO. 2 , PEACE RIVER TOWN




[A] SECTION AT CHAINAGE 1204 + 50




[B] SECTION AT CHAINAGE 1203 + 50

FIGURE IV-4 PROJECT AREA 'C' – SECTIONS ALONG HIGHWAY NO. 2, PEACE RIVER TOWN.

### 4.2.2 Stratigraphy

Figures IV-5, IV-6, and IV-7 show the observed stratigraphy along the cut slopes in the study area. The Figures show that the soil deposits are quite heterogeneous. Sand and silt pockets are prevalent and are, in places, associated with seepage as shown on Plates I and II.

Some difficulty was experienced in distinguishing between the tills and lacustrine deposits in Project Area A. However, Figure IV-5 shows the observed boundaries between various soil types. The eroded notch shown in Figure IV-5 and Plate I, in the outwash sand, is due to the seepage and surface water run-off.

Project Area B consists of tills and lacustrine deposits which are very silty and sandy in appearance. There is no seepage visible on the surface in this area. It is hard to establish if the deposits are in place or are debris of old slides. Figure IV-6 shows the stratigraphy as exposed along the cuts.

Project Area C appears erratic in nature. Plate II shows the seepage at various places in the area. Figure IV-7 shows the extent of observed cracks and discontinuities in the strata. Thick deposits of till are found which change abruptly into stratified lake deposits. Intactness of the deposits is questionable. The lacustrine deposits at about chainage 1204+00 could perhaps be designated as

varved. Plate III shows the appearance of a sample before and after drying. The deposits at this chainage are more silty at greater depths and the clay content increases as the depth decreases.

The deposits of the cuts on the south cut slopes in the Project Area C exhibit a tendency to disintegrate readily and slough out, as shown on Plate IV.

All the slopes on which the stratigraphy was mapped were 1 on 2 slopes.

## 4.3 Sampling

Samples were collected during the detailed geological mapping of the cut slopes. The various types of samples collected were:

(i) Undisturbed block samples for triaxial and consolidation tests.

(ii) Undisturbed shelby tube samples for triaxial tests.

(iii) Bag samples for classification tests.

Block samples were selected block of clays and tills from which triaxial specimens were cut. The samples were wrapped with polythene at the site and then transported to the laboratory. The location of each block sample was chosen and the position and orientation was carefully noted.

Three inch diameter shelby tube samples were taken in areas that appeared relatively softer to work, because a manual drop-hammer was employed to drive the tube into the soil.

The bag samples were collected at random.

Various positions of the samples taken are shown on the geological sections in Figures IV-5, IV-6 and IV-7.

In the laboratory, the block samples and the shelby tube samples were waxed and placed in the moisture room at about 70% humidity until required for testing.

Undisturbed test specimens for triaxial test were carefully prepared from the blocks using a wire saw and shaving knives. First, the blocks were sawn up into smaller rectangular prisms. The prism was then mounted in a rotary soil trimmer and trimmed to a circular cross section by gradually shaving away the surplus material with the thin wire saw. The specimens were then cut to the required length of three inches. In preparing the cylindrical specimens, particular attention was paid to the direction of the laminations. Most of the samples were with the laminations at right angles to the cylindrical axis. Since the laminations in-situ were mainly horizontal, a test specimen is described here as vertical when its cylindrical axis and the direction of the major principal stress is normal to the plane of laminations. A horizontal specimen thus has the cylindrical axis and the major principal stress parallel to the plane of laminations.

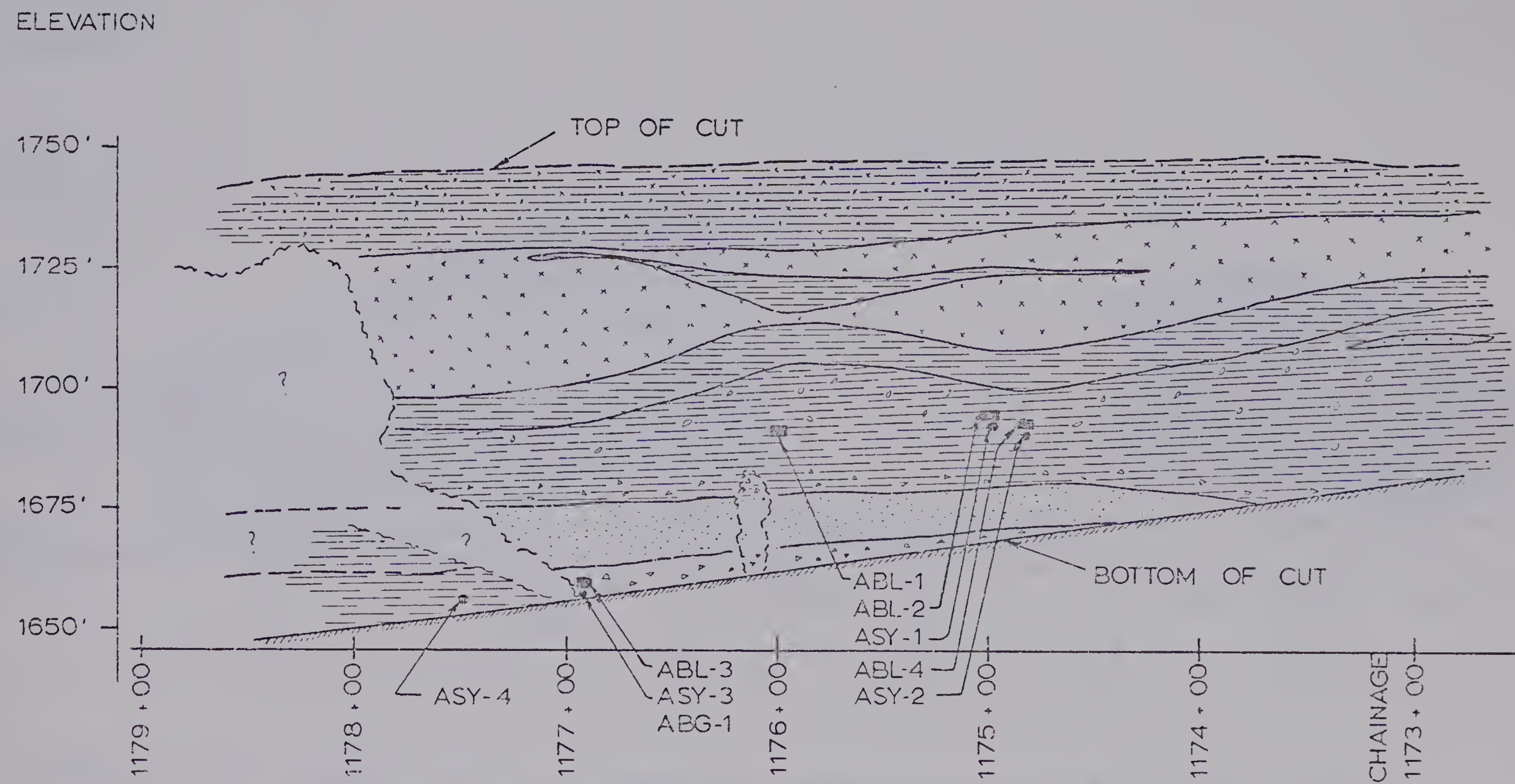



FIGURE IV.5 - PROJECT AREA 'A', OBSERVED STRATIGRAPHY ON 1 : 2 BACKSLOPES.

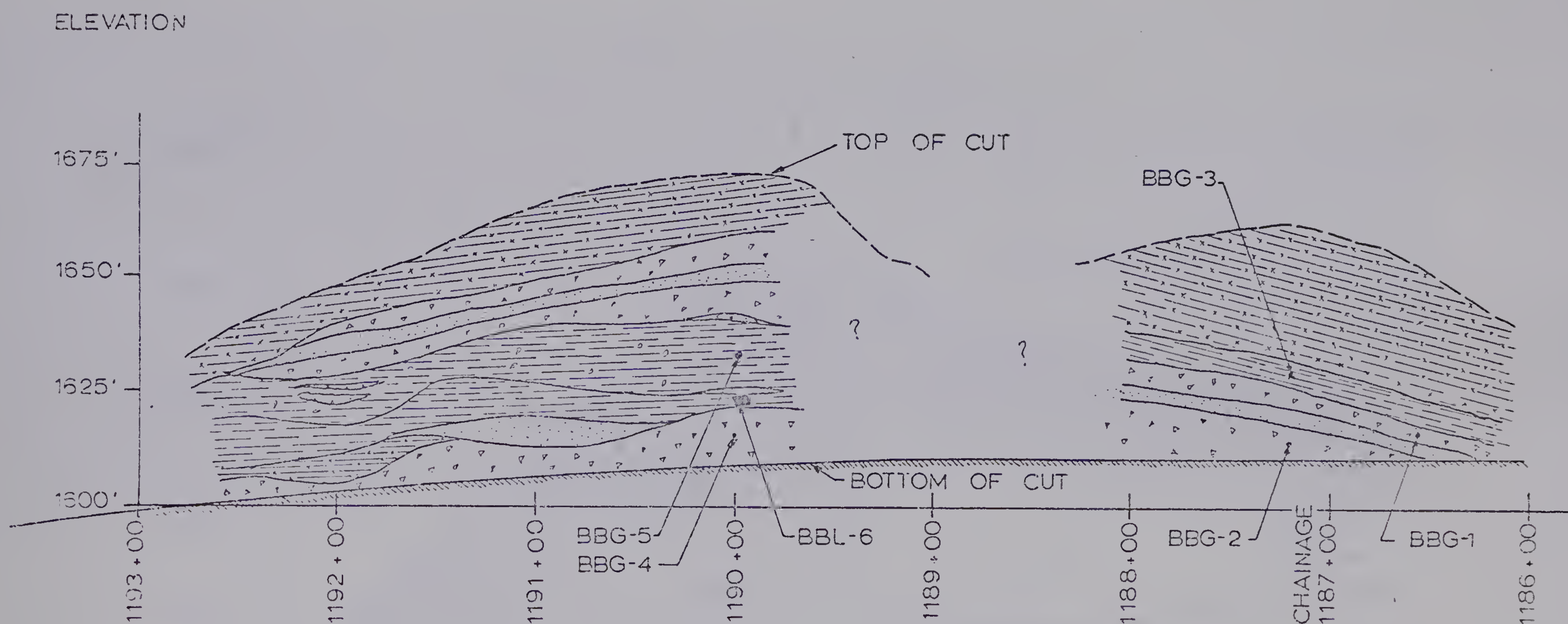



FIGURE IV.6 - PROJECT AREA 'B', OBSERVED STRATIGRAPHY ON 1:2 BACKSLOPES

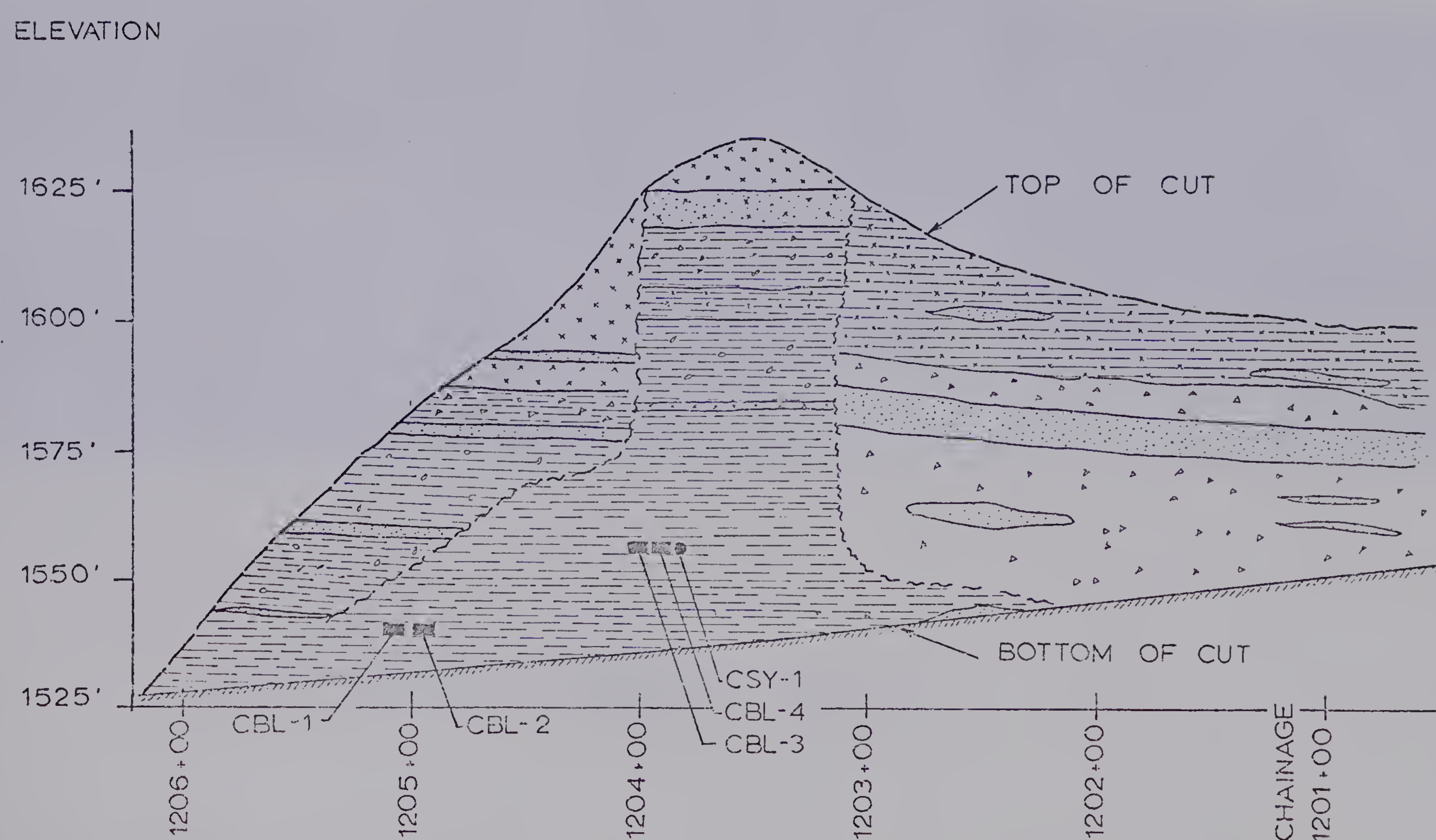



FIGURE IV. 7-PROJECT AREA 'C'. OBSERVED STRATIGRAPHY ON 1:2 BACKSLOPES


PLATE I - PROJECT AREA 'A'


PLATE II - PROJECT AREA 'C'

SAMPLE AT NATURAL
MOISTURE CONTENT


SAMPLE AFTER DRYING

PLATE III - VARVED CLAY FROM PROJECT AREA 'C'



PLATE IV - PROJECT AREA 'C' [SOUTHSIDE OF THE CUT]

# CHAPTER V

# LABORATORY TESTS PROCEDURES AND RESULTS

## 5.1 Testing Program

To obtain the geotechnical properties of the various deposits, a series of laboratory tests were carried out on both undisturbed and remoulded samples. These comprised:

(a) Specific gravity of soil solids.

(b) Grain size distribution analysis.

(c) Atterberg limits.

(d) Consolidation tests.

(e) Triaxial compression tests -

  (i) Consolidated undrained tests with pore pressure measurements

  (ii) Consolidated drained tests.

Mineralogical analyses by x-ray diffraction were provided by the Research Council of Alberta.

## 5.2 Testing Procedures and Results

Standard geotechnical tests were performed in the laboratory. In the following section a brief discussion of the laboratory techniques and the results obtained are presented.

### 5.2.1 Classification of Soils

Classification tests were carried out on soils initially at their natural moisture contents.

(a) Particle Size Distribution

Figure V-1 is the summary of the grain size distribution curves for the bulk samples of the soils tested. The percentage of clay sizes varied from 50 to 84. A small percentage of gravel was found associated with all the samples of the Project Area A. The clay size in the upper strata of the Project Area C (sample CBL-3) was found to be highest (up to 84 percent).

The grain size analyses were carried out by means of an hydrometer. The pretreatment of the sample involved treating the soil with 12% $H_2O_2$ and allowing it to stand overnight. About 25 c.c. of 6% calgon were used as the dispersion agent.

(b) Atterberg Limits, Mineralogy

Soils at their natural moisture contents were soaked for at least 12 hours before determining the Atterberg Limits. The gravels encountered were discarded in the tests. The procedures adopted were as given by ASTM (1966) and for liquid limit tests a Casagrande type grooving tool was used.

The test results are summarized in the Table V-1. The liquid limit of the soils tested varied from 50 to 84 percent. Figure V-2 shows the plasticity chart. The range

TABLE V-1

SUMMARY OF INDEX PROPERTIES

| Location | Sample | Description | Unit Weight lb/ft$^3$ | Natural Moisture Content w % | Liquid Limit LL % | Plastic Limit PL % | Plasticity Index PI % | Liquid. Index $I_L$ | % Sand | % Silt | % Clay | Act-ivity | Specific Gravity of Soil Solids $G_s$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Project Area A | ABL-2 | Grey Clay (Varved Silty bands. | 119 | 35 | 74 | 32 | 42 | .07 | 14 | 36 | 50 | 0.84 | 2.74 |
| | ABL-3 | Grey Clayey Till | 135 | 14 | 37 | 16 | 21 | -.1 | | | | | 2.70 |
| | ABG-1 | Grey Clayey Till | | | 52 | 24 | 28 | | 2.5 | 54.5 | 43 | 0.56 | |
| | ASY-4 | Grey Clay From Slide Area | | | | | | | 1.0 | 44 | 55 | | |
| Project Area B | BBL-6 | Silty Clay (Lake?) | | | 50 | 24 | 26 | | | | | | |
| Project Area C | CBL-1 | Laminated Lake Sediments | 120 | 28 | 61 | 26 | 34 | .04 | 7 | 46 | 47 | .56 | |
| | CBL-2 | Laminated Lake Sediments | | | | | | | | | | | 2.70 |
| | CBL-3 | Dark Grey Lake Clay | 121 | 32 | 84 | 34 | 50 | -.05 | - | 14 | 84 | .42 | 2.73 |
| *Pennell | "Pre-Till Clay" | | 118 | 32 | 73 | 27 | 46 | .11 | 4 | 31 | 65 | .7 | 2.75 |
| | "Post-Till Clay" | | 119 | 27 | 60 | 22 | 38 | .13 | 3 | 40 | 57 | .7 | 2.77 |

*After Pennell, 1969

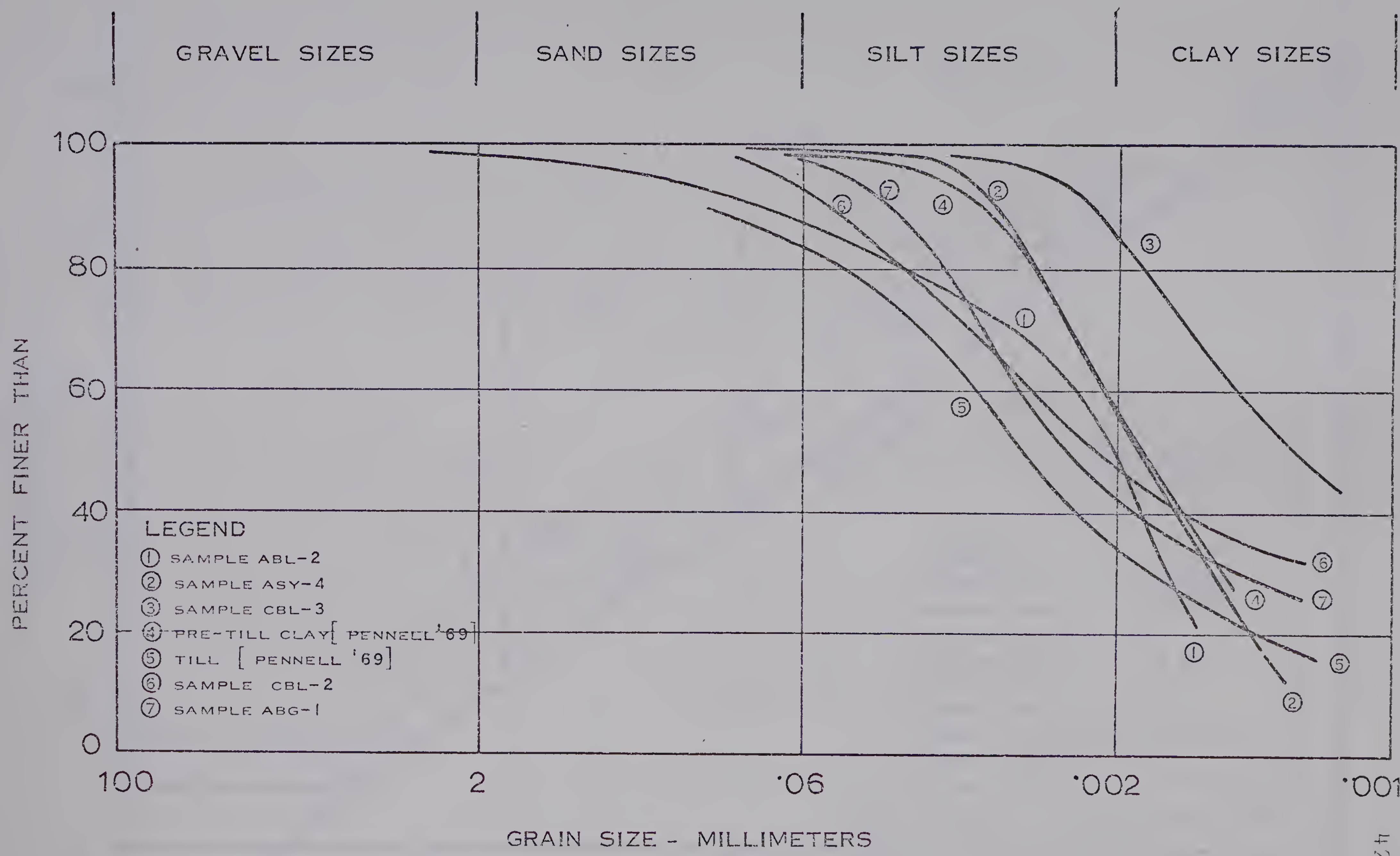



FIGURE V-1 GRAIN SIZE DISTRIBUTION CURVES SUMMARY

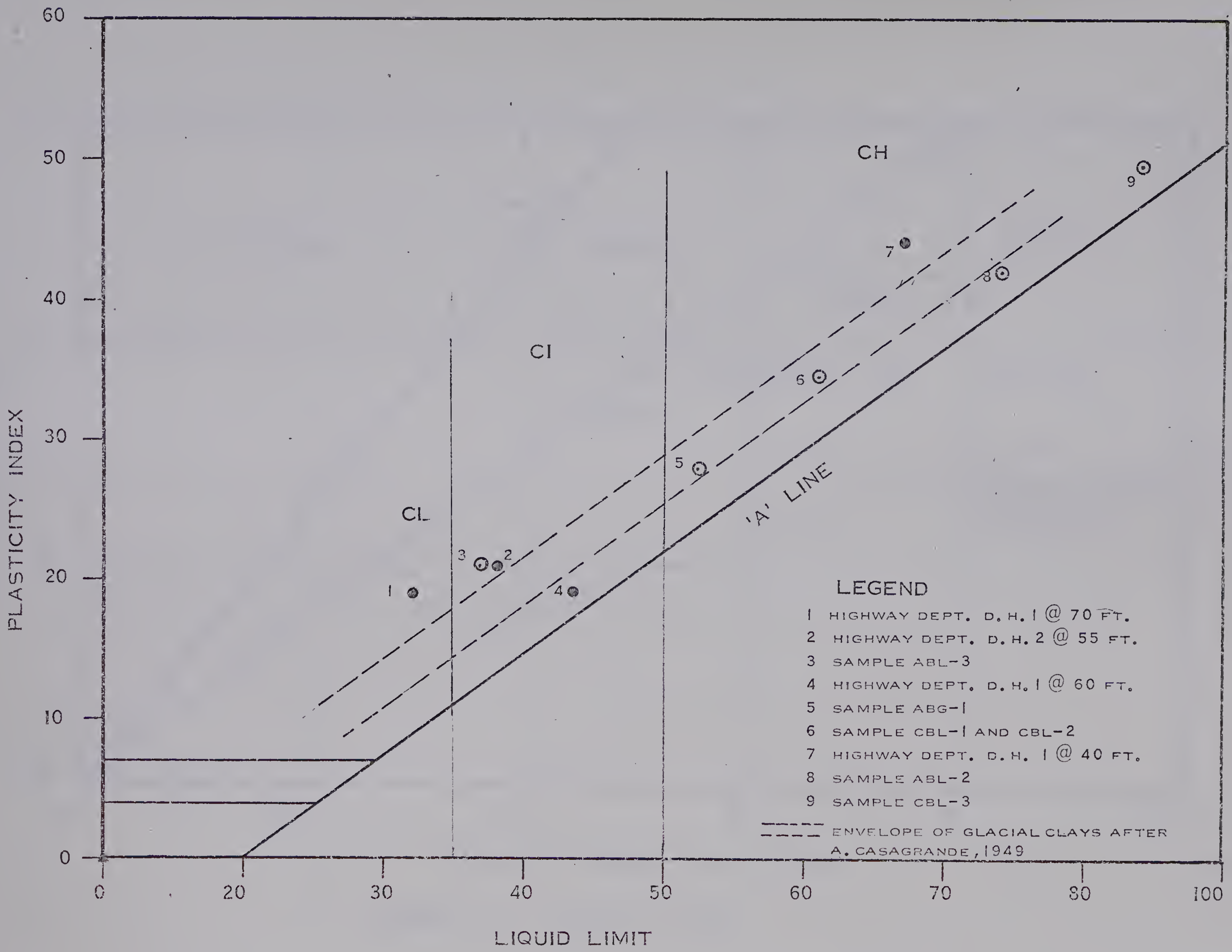



FIGURE V-2 PLASTICITY CHART

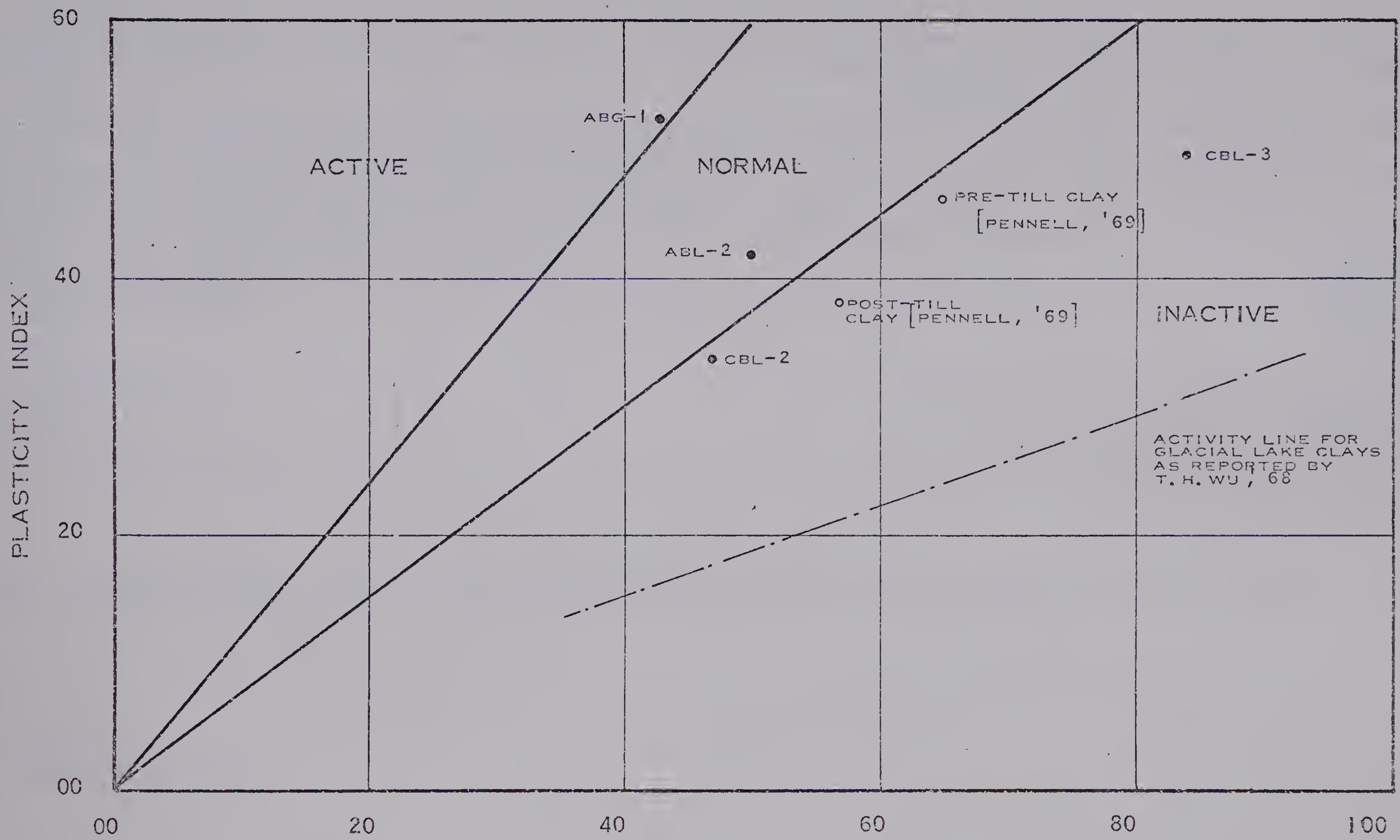



FIGURE V-3 ACTIVITY CHART

TABLE V-2

MINERALOGICAL COMPOSITION OF CLAY SIZES

| Sample | | | Illite | Montmo-rillonite | Chlorite | Kaolinite | Remarks |
|---|---|---|---|---|---|---|---|
| ASY | Project Area A | Grey Clay Highly Plastic | 62% | 11% | 4% | 23% | |
| CBL-3 | Project Area C | Stratified Grey Clay | 64% | 10% | 3% | 23% | |
| ABL-2 | Project Area A | Clay Till | 80% | - | | 20% | 20% Chlorite and/or Kaolinite |
| *"Pre-Till Clay" | | | 58% | 17% | | 25% | 25% Chlorite and/or Kaolinite |
| *"Post-Till Clay" | | | 58% | 22% | | 20% | 20% Chlorite and/or Kaolinite |

Note: Traces of Quartz, Feldspar, Calcite and Dolomite were also found present in the clay fraction.

* After Pennel, 1969

of plasticity value for glacial lake clays (after A. Casagrande, 1948) has also been shown on Figure V-2.

Figure V-3 shows a plot of the clay fraction versus the plasticity index (Activity Chart).

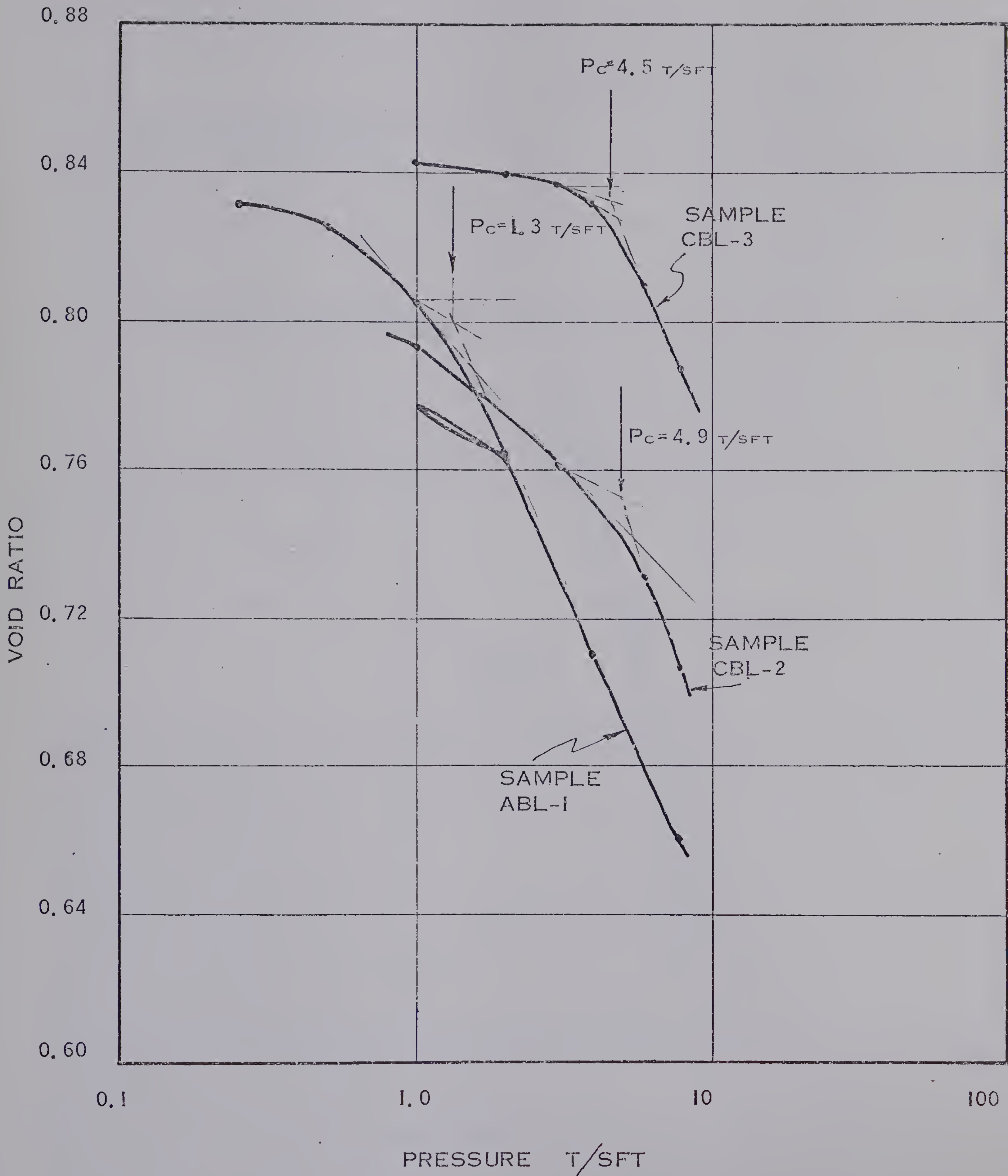
The mineralogical findings are listed in Table V-2. The soils are predominately illite.

### 5.2.2 Consolidation

One-dimensional and triaxial consolidation tests were performed on undisturbed samples at their natural moisture contents.

Consolidation in an oedometer was carried out on samples 2.5 inches in diameter and 0.75 inch thick, utilizing a fixed ring type oedometer. Plate V shows the types of consolidometers used. In the tests, sample ABL-2 was flooded before loading, whereas in rest of the one-dimensional consolidation tests the samples were flooded after they had been subjected to a pressure approximately equal to the present (preconstruction) overburden pressures. The e-log $p$ curves obtained are shown in Figure V-4. Table V-3 summarizes the consolidation tests results. Shown on the e-log $p$ curves are the present (preconstruction) overburden pressures.

The preconsolidation pressures $p_c$, are calculated from e-log $p$ curves employing Casagrande's graphical technique (Casagrande, 1932).




F FIGURE V-4. OEDOMETER CONSOLIDATION RESULTS FOR SAMPLES ABL-1, CBL-2, AND CBL-3.

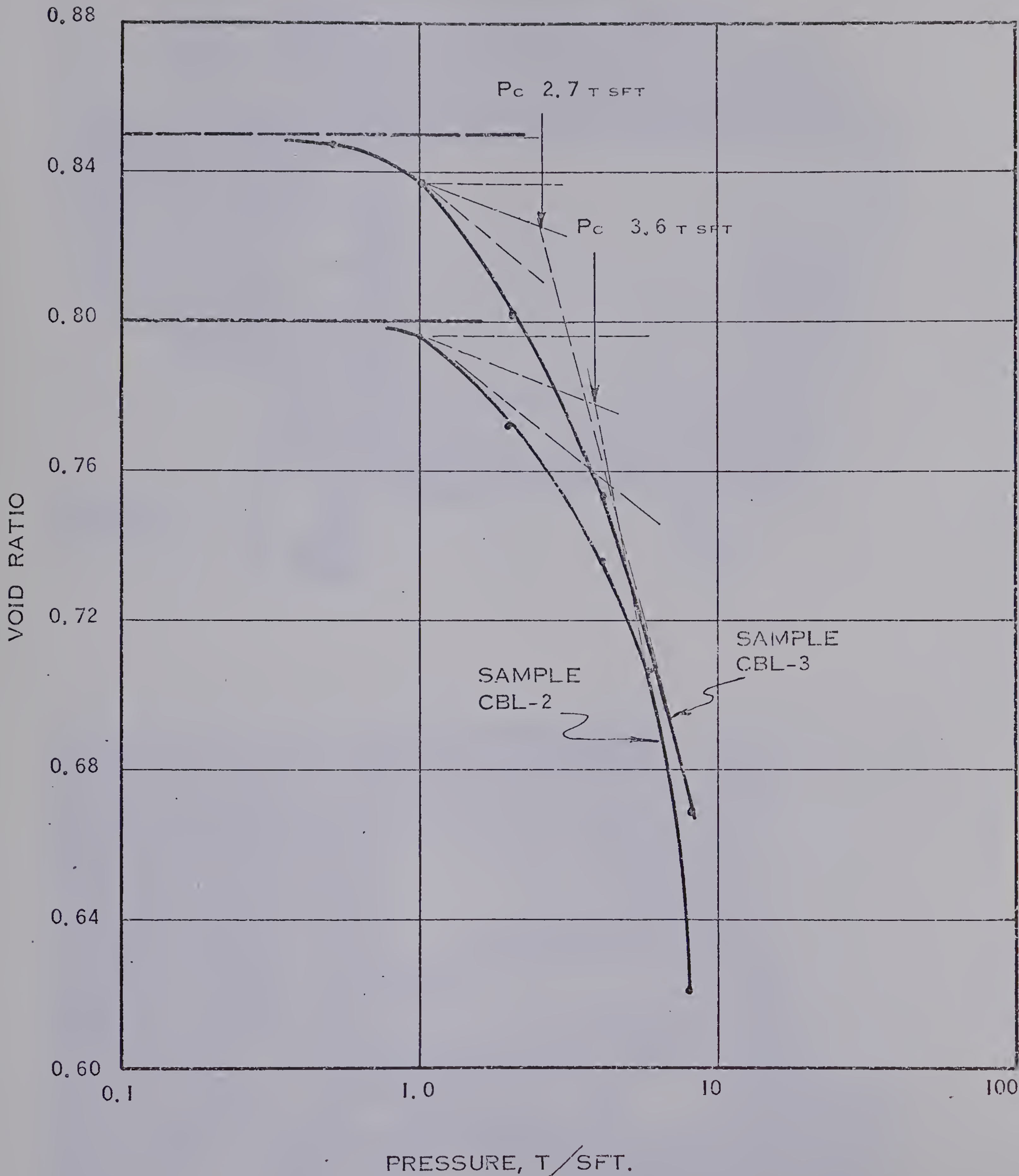



FIGURE V-5 TRIAXIAL CONSOLIDATION RESULTS FOR SAMPLES CBL-2 AND CBL-3

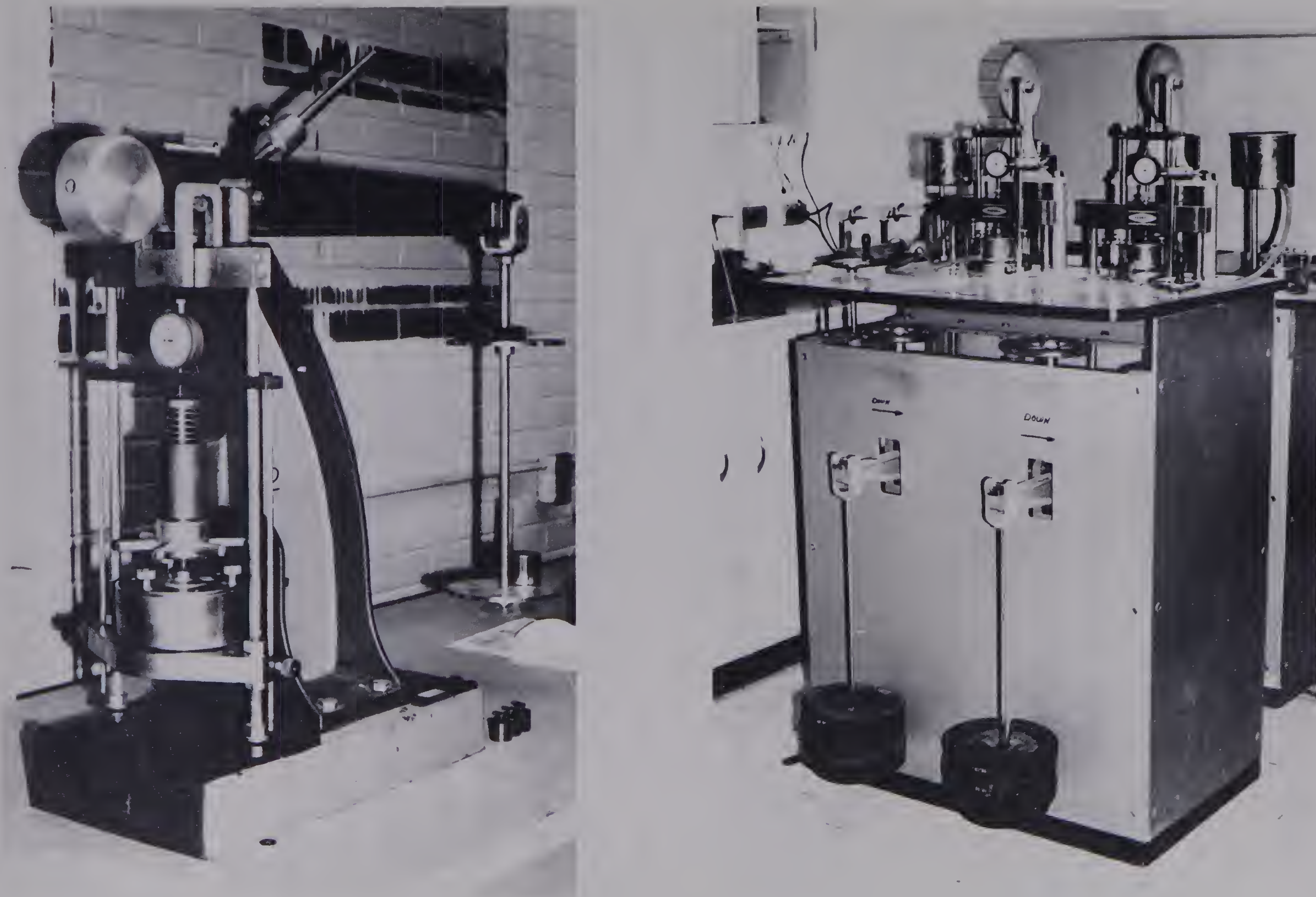



PLATE V - ONE DIMENSIONAL CONSOLIDATION APPARATUSES

Triaxial consolidation tests were carried out under a back pressure of 30 psi. However, no back pressures could be applied for higher loads, because of the limitations of the loading system. Figure V.5 shows the triaxial consolidation test results on samples CBL-2 and CBL-3. The plots show the relationship of volume change to the logarithm of confining pressures.

### 5.2.3 Shear Tests

Triaxial compression tests were performed to study the shear strength characteristics of the soils. Consolidated undrained and consolidated drained tests were carried out on undisturbed samples. A limited number of tests were performed on remoulded samples. In all the triaxial tests, pore pressure measurements were taken utilizing a pressure transducer. The applied deviator loads in the tests were measured by a load cell. A linear voltage differential transformer (LVDT) was used for measuring the vertical displacements. Details of the instrumentation are given in Appendix B. The specimen size for the tests was 1.5 inches in diameter and 3 inches long. Filter paper strip side drains were used.

A back pressure of 30 psi was used in all the tests. The samples were tested under 38, 46 and 55 psi cell pressures, that is 8, 16 and 25 psi effective pressure. The procedures followed were those given by Bishop and

Henkel (1964). Corrections for the filter strip and the membrane have been applied to the loads at failure.

Most of the undisturbed samples of clays failed at about 5 to 7% strain. A failure strain of 12% was used for the till samples of the Project Area A (sample ABL-3). The remoulded samples were hand kneaded and compacted under 10 psi by means of a kneading compaction machine. An arbitrary failure strain of 10% was taken for the remoulded samples.

Failure conditions were taken as those at the maximum deviator stress. Figures C-1 to C-8 in Appendix C show the stress-strain relationships. The Mohr circles for various soils are given in Figures V-6 through V-11. Table V-4 gives the summary of shear strength tests. The various strength parameters obtained are listed in Table V-5.

To observe the behaviour of the soils under various conditions of loading stress-paths have been plotted on modified Mohr Diagrams as shown in Figures V-12 through V-17.

A reversal direct shear test on a pre-cut plane for sample CBL-2 was performed. The normal load applied was 25 psi. The residual friction angle thus obtained is based on single shear test and is found to be 10° as shown in Figure V-12.

TABLE V-4

SHEAR STRENGTH TESTS SUMMARY

| Sample | Type of Test | $I_L$ | $e_n$ | $W_n$ | $W_f$ | $\bar{\sigma}_c$ Psi | % $\varepsilon_f$ | $(\sigma_1-\sigma_3)_f$ | $A_f$ | B | Remarks |
|---|---|---|---|---|---|---|---|---|---|---|---|
| ABL-3 | $CU_u$ | -.1 | .44 | 14 | 13.9 | 25 | 12 | 52 | .07 | .98 | Grey Clayey Till |
| | $CU_u$ | | | 15.9 | 16.6 | 8 | 12 | 30.65 | .238 | .95 | |
| ABL-1/ ABL-2 | $CU_u$ | | .91 | 34.5 | 32 | 25 | 6.5 | 25.4 | .38 | .76 | Undisturbed Varved Clay |
| | $CU_u$ | .07 | .84 | 23 | | 16 | 3.0 | 24.6 | .28 | .94 | |
| | $CU_u$ | | | 22 | 22 | 8 | 4.5 | 14.27 | .173 | .9 | |
| ABL-1/ ABL-2 | $CU_R$ | | | 33 | 34 | 16 | 8.5 | 11.29 | .454 | .9 | Varved Clay Remoulded |
| | $CU_R$ | | | 33 | 35.11 | 25 | 5 | 24.2 | .35 | .86 | |
| | $CU_R$ | | | | | 8 | 10 | 10.8 | .237 | | |
| CBL-3 | $CU_u$ | -.05 | .86 | 31.3 | 36.4 | 25 | 6.2 | 31.5 | .17 | .87 | Stratified Highly Plastic Clay |
| | $CU_u$ | | .85 | 31.5 | 38.5 | 16 | 5 | 22.5 | .18 | .75 | |
| | $CU_u$ | | .84 | 31 | 39.3 | 8 | 6 | 15.8 | .14 | .876 | |
| CBL-1 | $CU_R$ | | .83 | 29 | 28.5 | 25 | 10 | 30.95 | .15 | .78 | Varved Clay - Remoulded |
| | $CU_R$ | | .84 | 28 | 30.2 | 16 | 10 | 29.5 | .186 | .72 | |
| | $CU_R$ | | .83 | 28.5 | 31 | 8 | 10 | 20.0 | -.0095 | .79 | |
| ABL-2 | $CD_U$ | | .72 | 33 | 25.1 | 16 | 5.3 | 25.7 | - | .9 | Clayey Till |
| | $CD_U$ | .07 | .8 | 34 | 31 | 8 | 4.2 | 11.4 | - | .92 | |
| CBL-1 | $CU_u$ | | .8 | 29 | 26.2 | 25 | 6 | 50.2 | .032 | .87 | Horizontal Specimen. i.e. Cylindrical axis parallel to the laminations |
| | $CU_u$ | .04 | .81 | 28 | 28.2 | 16 | 4.3 | 34.86 | .08 | .89 | |
| CBL-1 | $CU_u$ | | .82 | 27.6 | 28 | 25 | 5 | 34.45 | .28 | .94 | Varved Clay |
| | $CU_u$ | .04 | .8 | 27.7 | 29 | 16 | 5.3 | 34.28 | .108 | .95 | |
| | $CU_u$ | | .81 | 27.6 | 29.5 | 8 | 3.8 | 20.2 | .118 | .90 | |

$CU_U$ - Consolidated Undrained Undisturbed
$CU_R$ - Consolidated Undrained Remoulded
$CD_U$ - Consolidated Drained Undisturbed
$CD_R$ - Consolidated Drained Remoulded
$I_L$ - Liquidity Index
$e_n$ - Natural Void Ratio
$w_n$ - Natural Moisture Content (%)

$\bar{\sigma}_c$ - Effective Cell Pressure
$\varepsilon_f$ - Strain at Failure
$(\sigma_1-\sigma_3)_f$ - Deviator Stress at Failure
$A_f$ - Skempton's Pore Pressure Parameter A at Failure
B - Skempton's Pore Pressure Parameter B
$w_f$ - Moisture Content at Failure (%)

TABLE V.5

SHEAR STRENGTH RESULTS

| Sample | Location | Type Test* | $W_u$ % | C' | Ø' | Remarks |
|---|---|---|---|---|---|---|
| ABL-2 | Project Area "A" | CUU | 33 | 0 | 27° | |
| ABL-2 | | CUU | 35 | 0 | 26.5° | |
| ABL-2 | | CDU | 35.6 | 0 | 26° | |
| ABL-2 | | CUR | 33 | 0 | 22° | |
| ABL-2 | | CDR | 37 | 0.8 | 20° | |
| CBL-1 | Project Area "C" | CUU | 28 | 1 | 30° | |
| CBL-1 | | CUR | 28.5 | 3.5 | 17° | |
| CBL-1 | | CUU | 18. | 1.5 | 28.5° | Horizontal Sample |
| CBL-3 | | CUU | 31 | 2.7 | 20° | |
| ABL-3 | Project Area "A" | CUU | 14 | 0 | 32° | |

*CUU - Consolidated-Undrained Undisturbed

CUR - Consolidated-Undrained Remoulded

CDU - Consolidated-Drained Undisturbed

CDR - Consolidated Drained Remoulded

c' - Effective Peak Cohesion in PSI

Ø' - Effective Peak Friction Angle

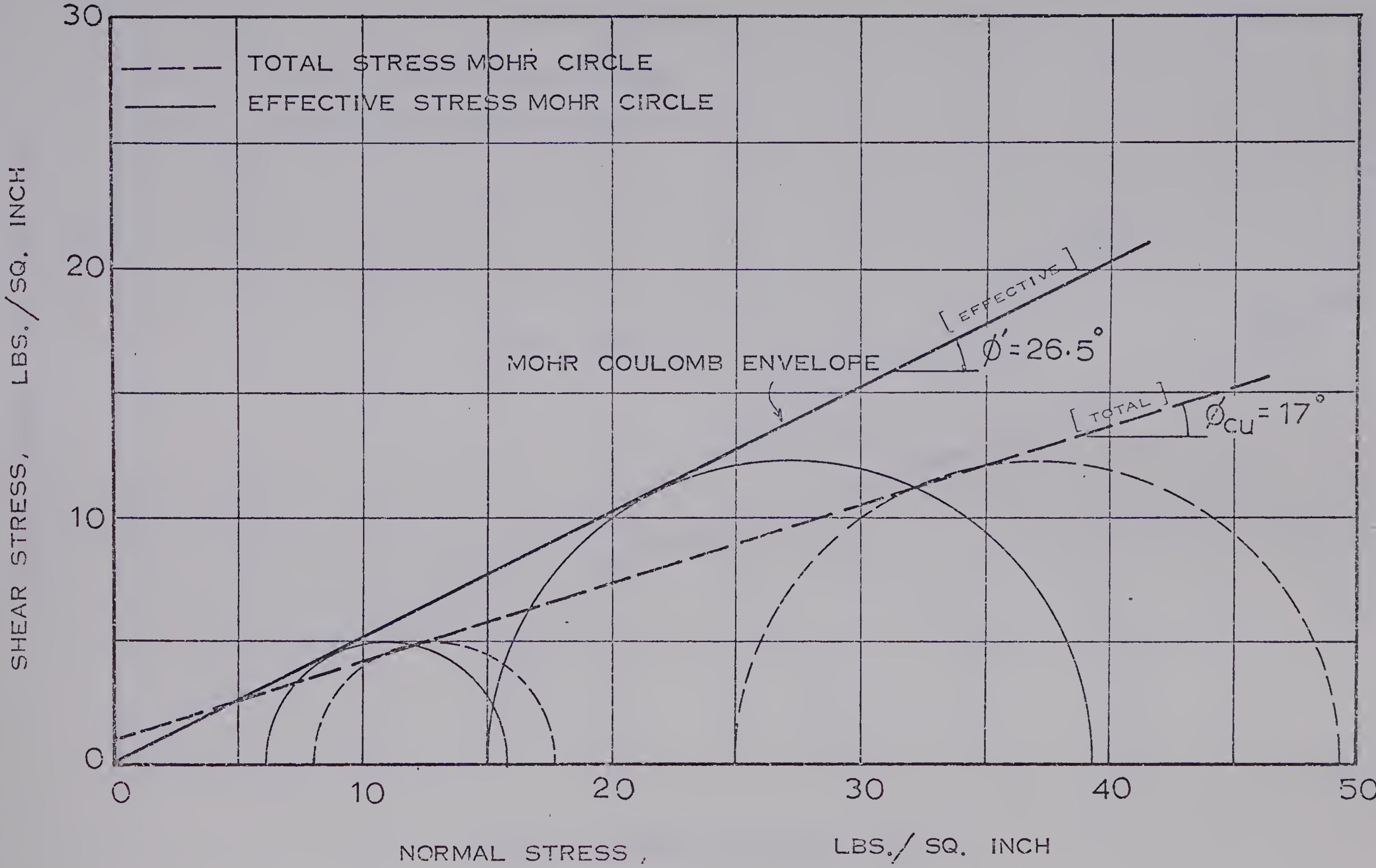



FIGURE V-6 MOHR CIRCLES AT FAILURE FOR C-U TESTS ON SAMPLE ABL-2 UNDISTURBED

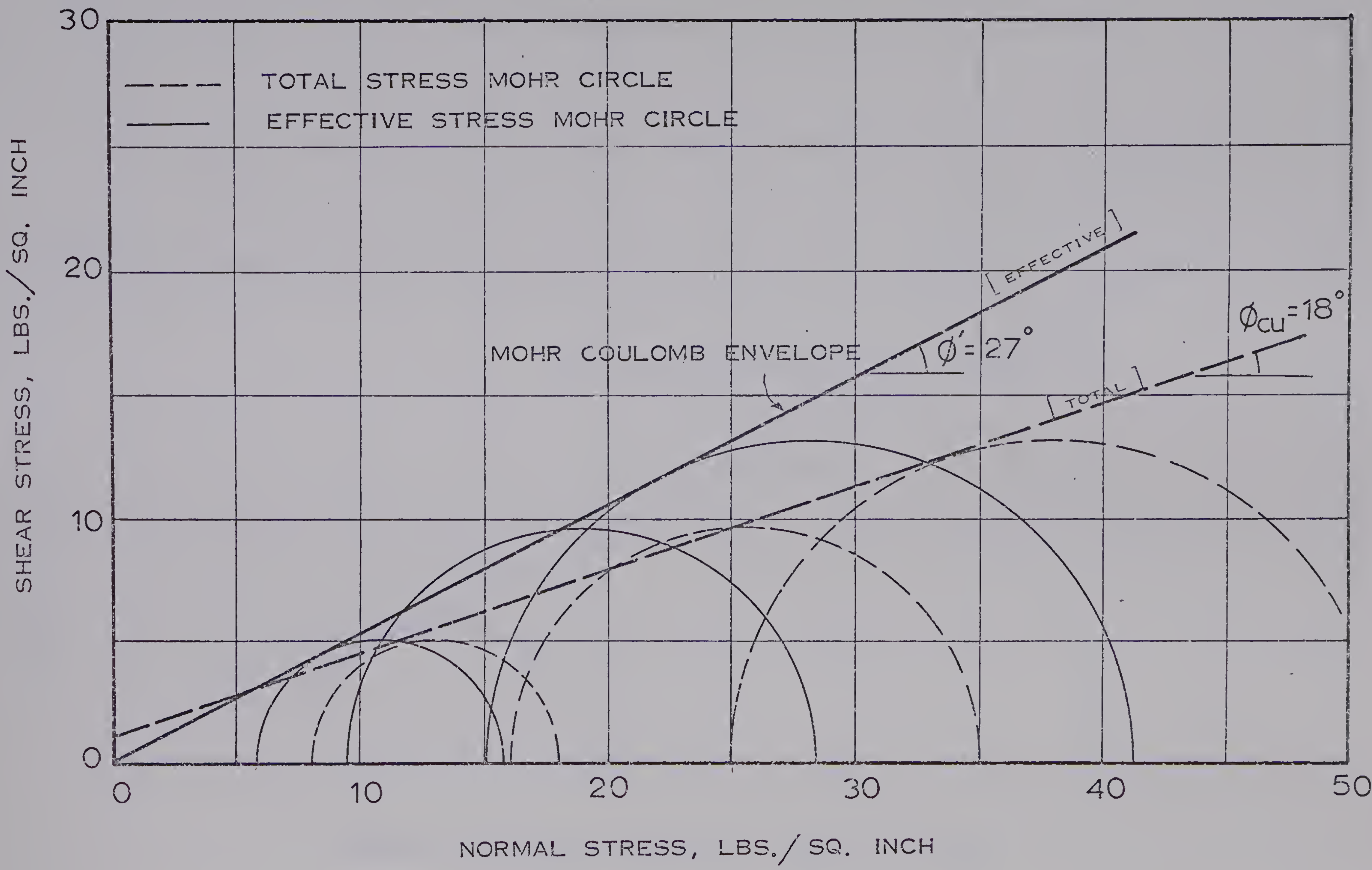



FIGURE V-7 MOHR CIRCLES AT FAILURE FOR C-U TESTS ON SAMPLE ABL-2 UNDISTURBED

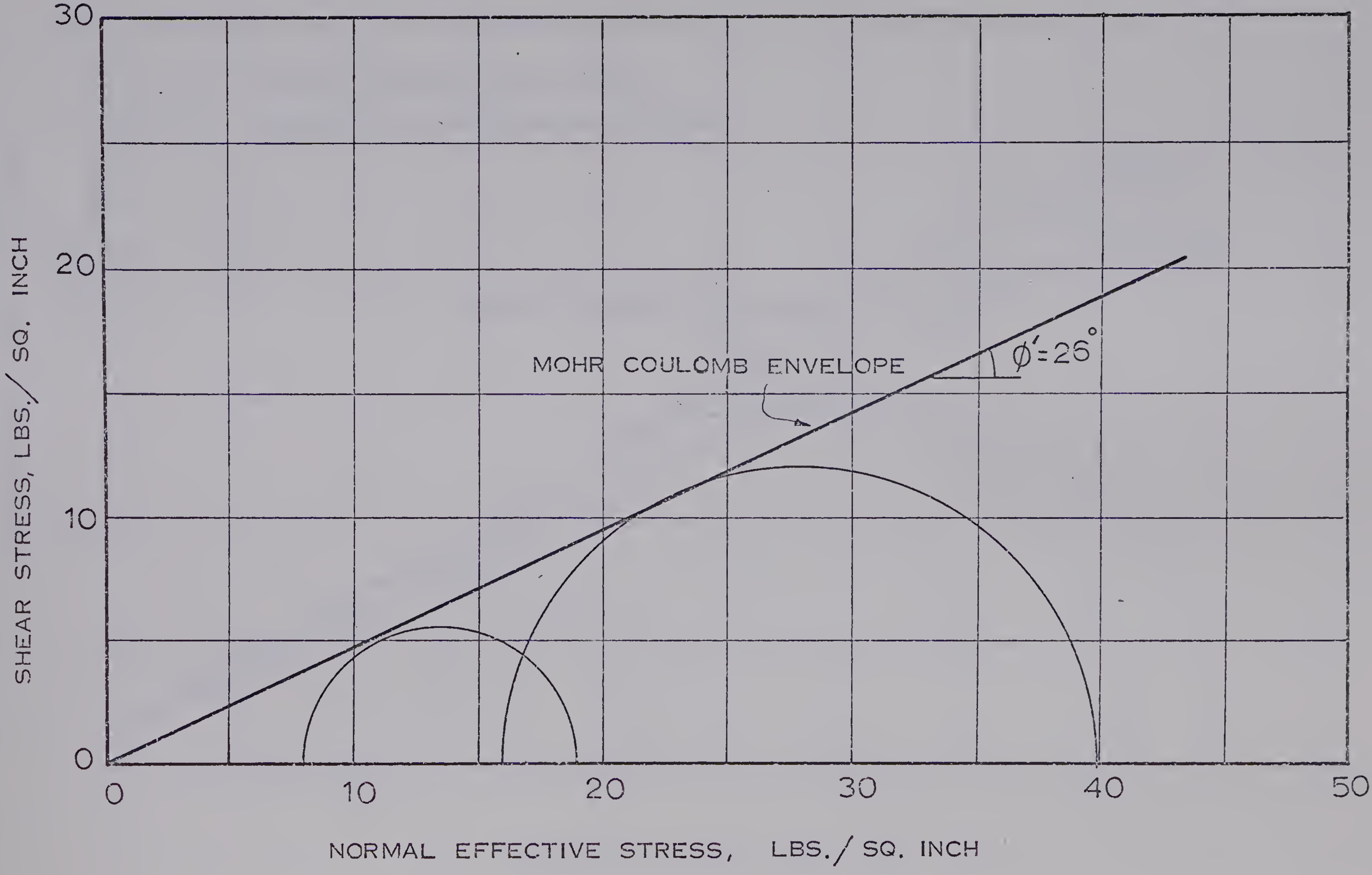



FIGURE V-8 MOHR CIRCLES AT FAILURE FOR C-D TESTS ON SAMPLE ABL-2 UNDISTURBED

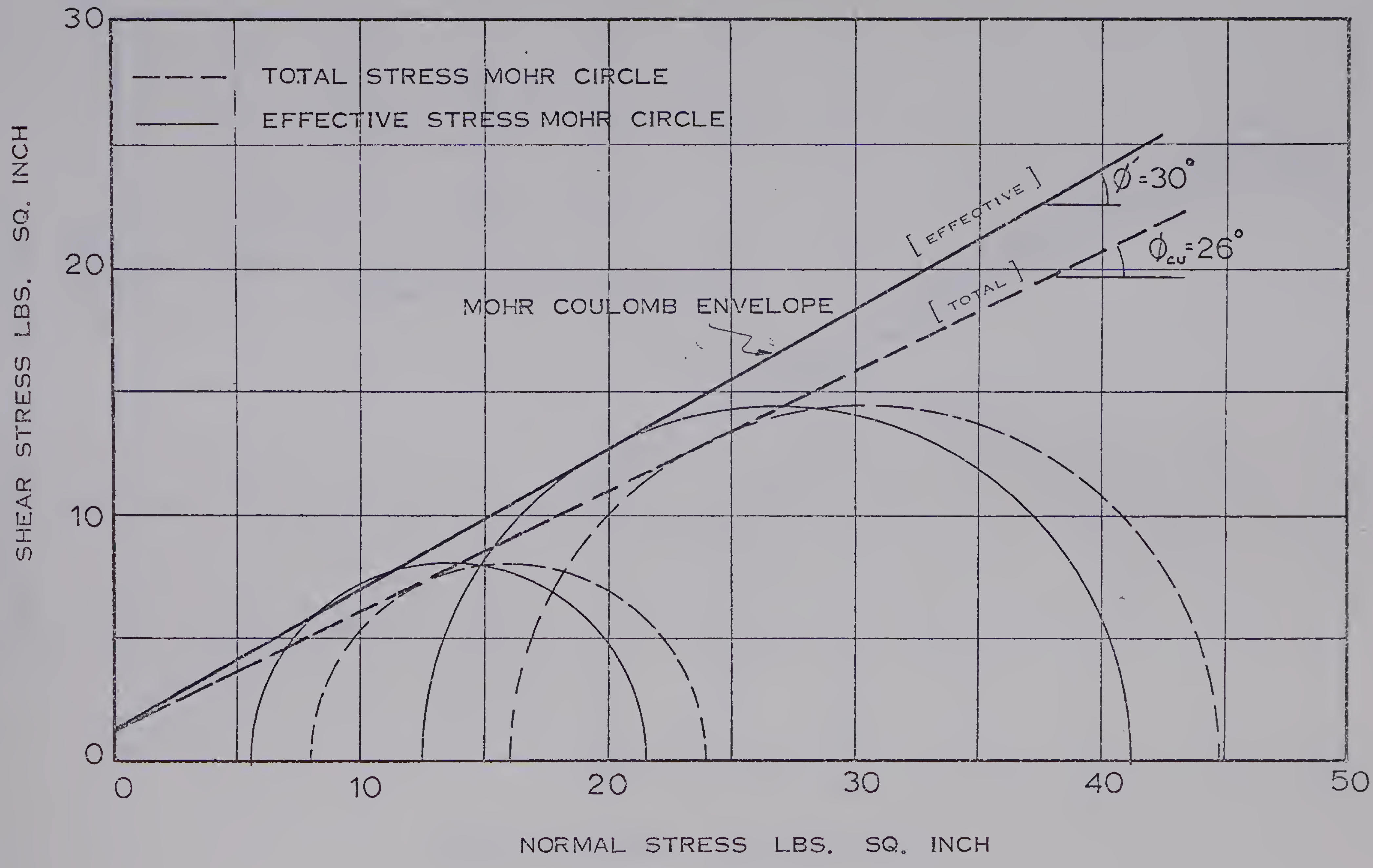



FIGURE V-9 MOHR CIRCLES AT FAILURE FOR C-U TESTS ON SAMPLE CBL-I UNDISTURBED

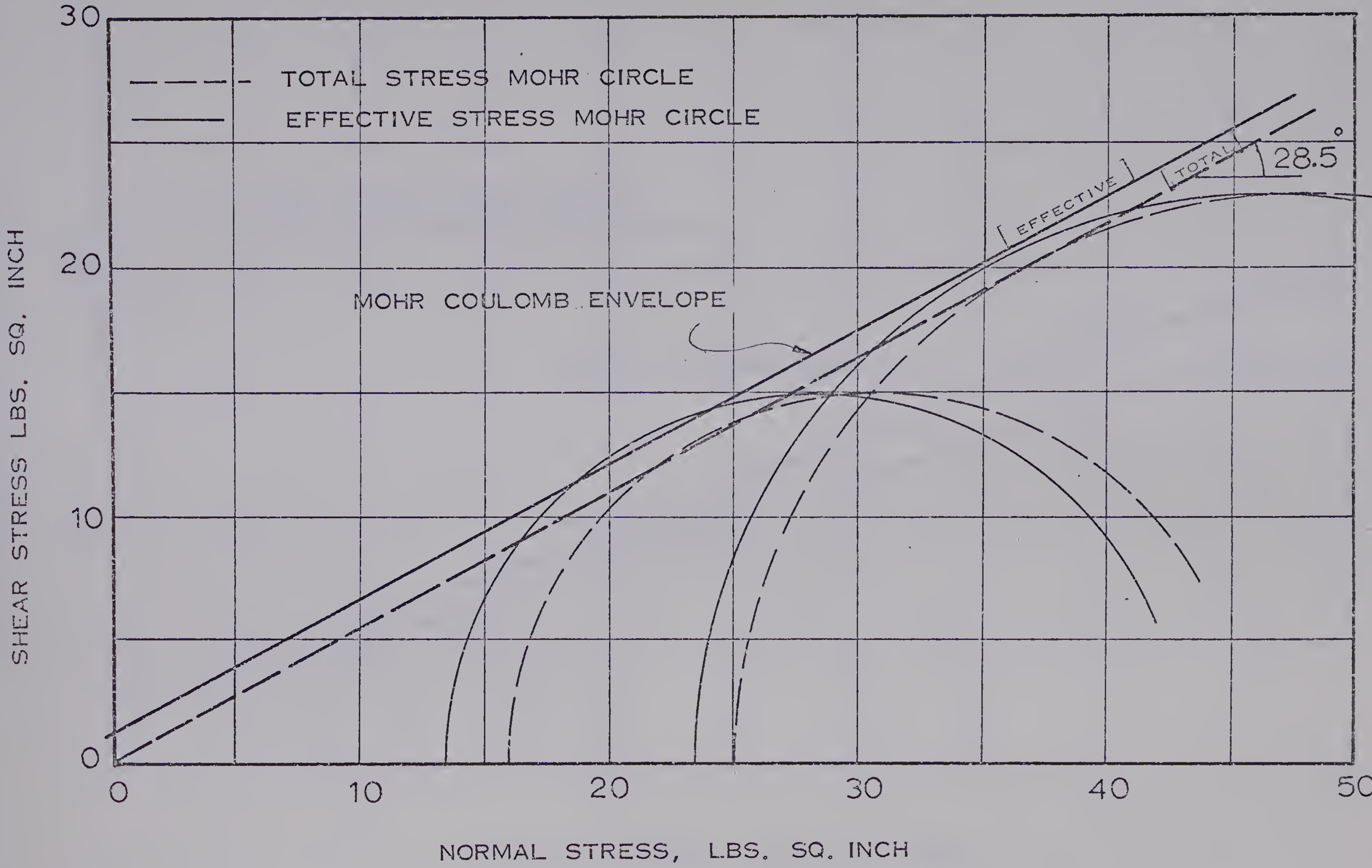



FIGURE V-10 MOHR CIRCLES AT FAILURE FOR C-U TESTS ON SAMPLE CBL-I UNDISTURBED, HORIZONTAL ORIENTATION.

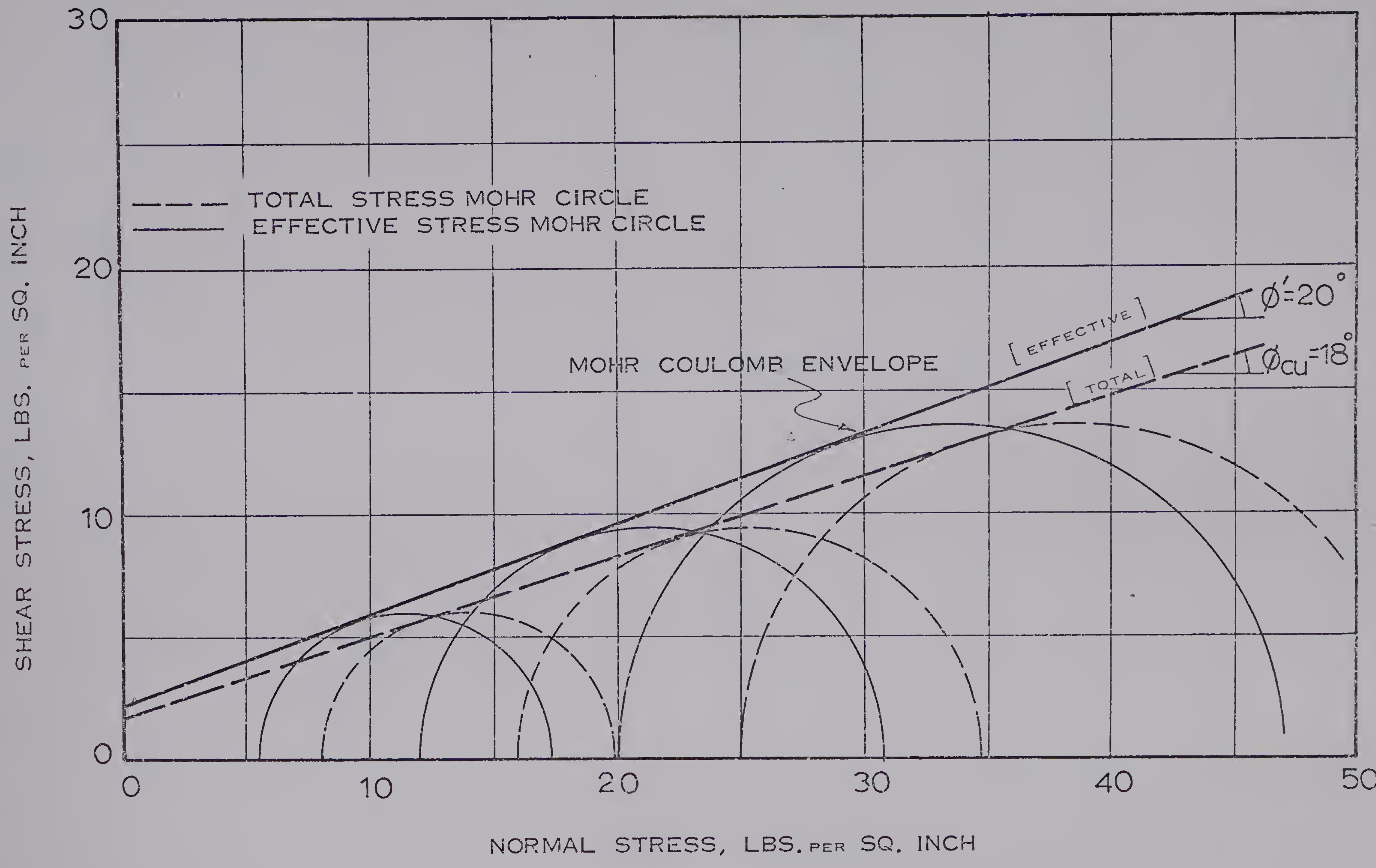



FIGURE V-11 MOHR CIRCLES AT FAILURE FOR C-U TESTS ON SAMPLE CBL-3 UNDISTURBED.

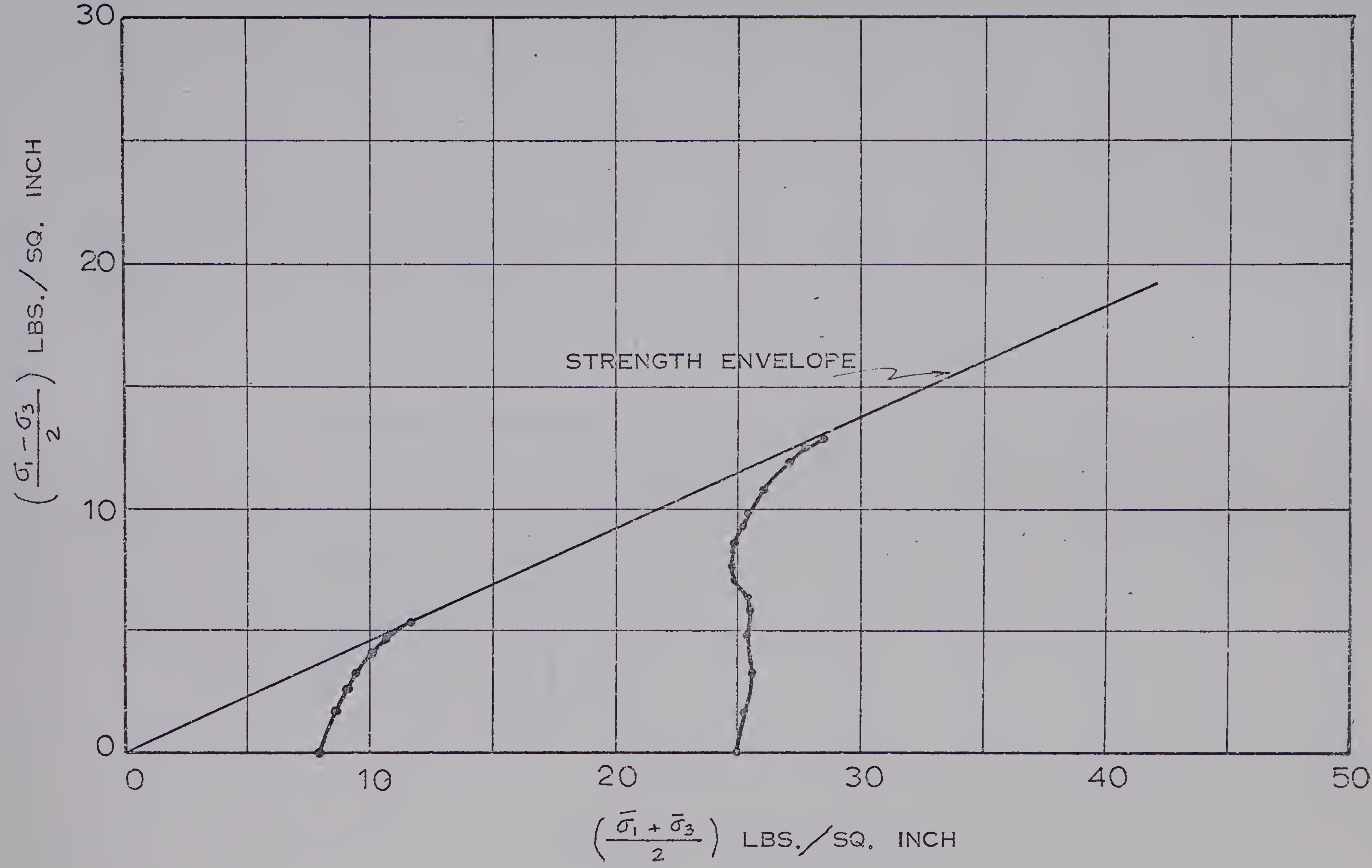



FIGURE V-12 EFFECTIVE STRESS PATHS AND STRENGTH ENVELOPE FOR C-U TESTS ON SAMPLE ABL-2 UNDISTURBED

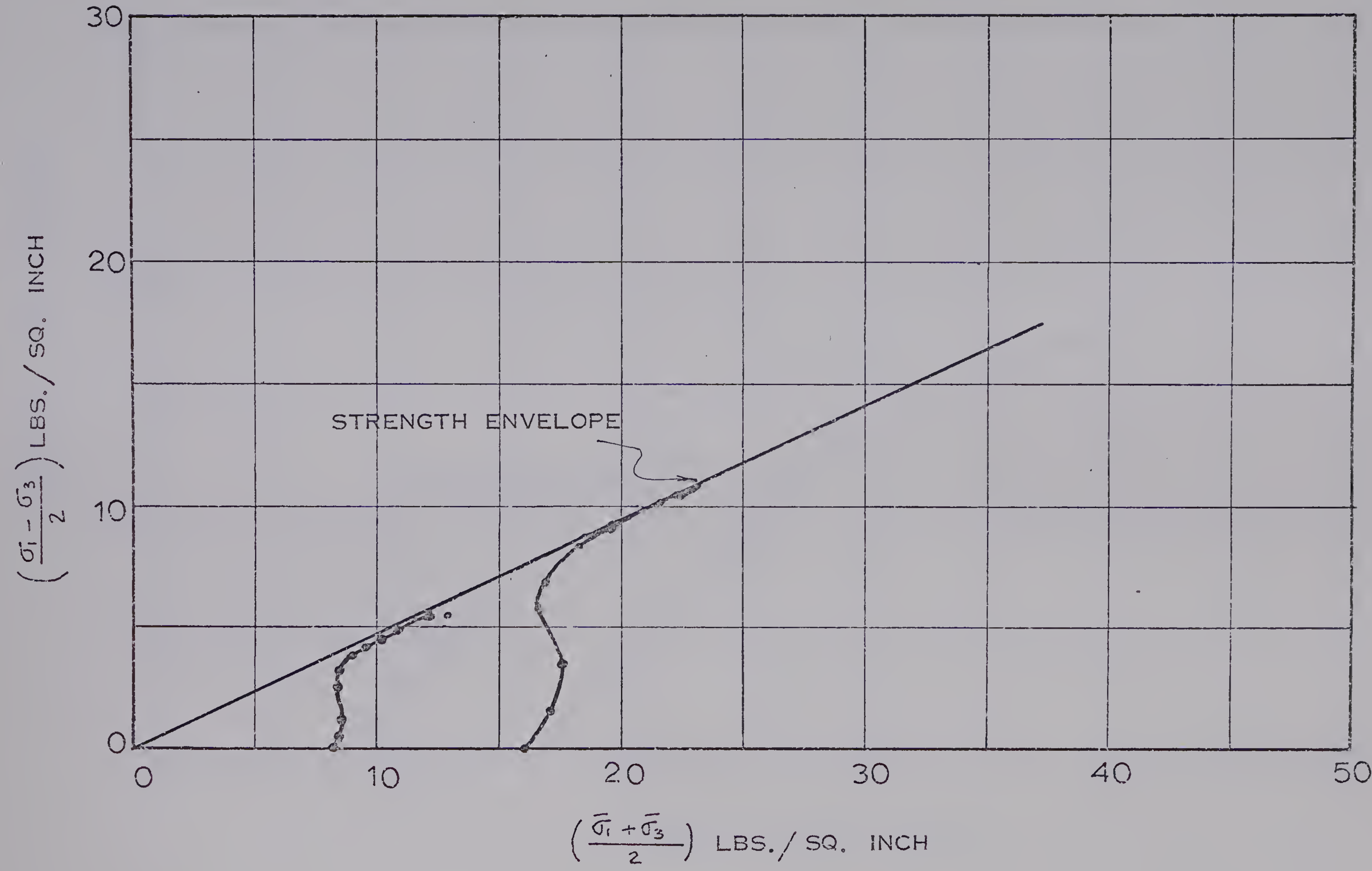



FIGURE V-13 EFFECTIVE STRESS PATHS AND STRENGTH ENVELOPE FOR C-U TESTS ON SAMPLE ABL-2 UNDISTURBED

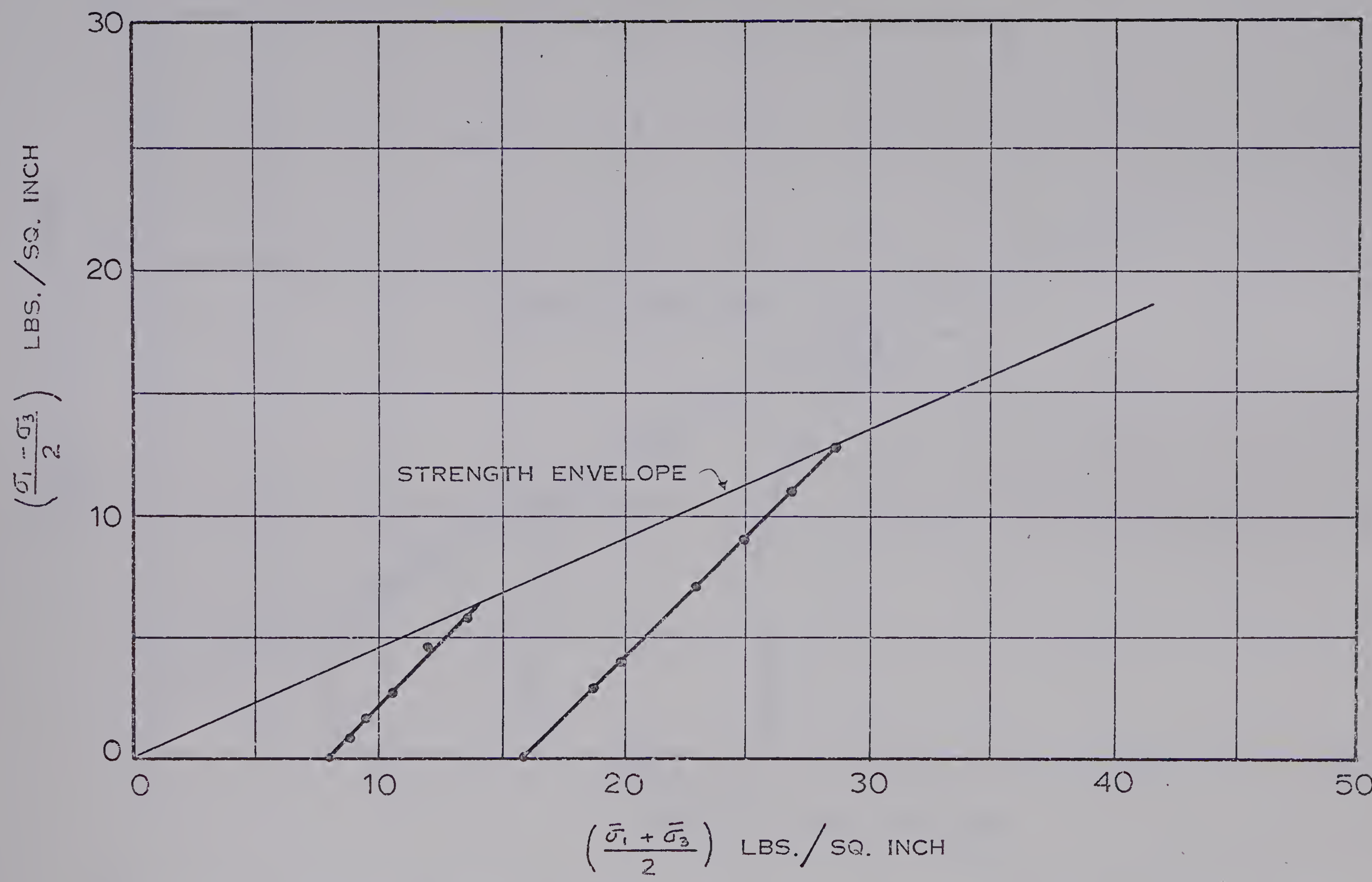



FIGURE V-14. EFFECTIVE STRESS PATHS AND STRENGTH ENVELOPE FOR C-D TESTS ON SAMPLE ABL-2, UNDISTURBED

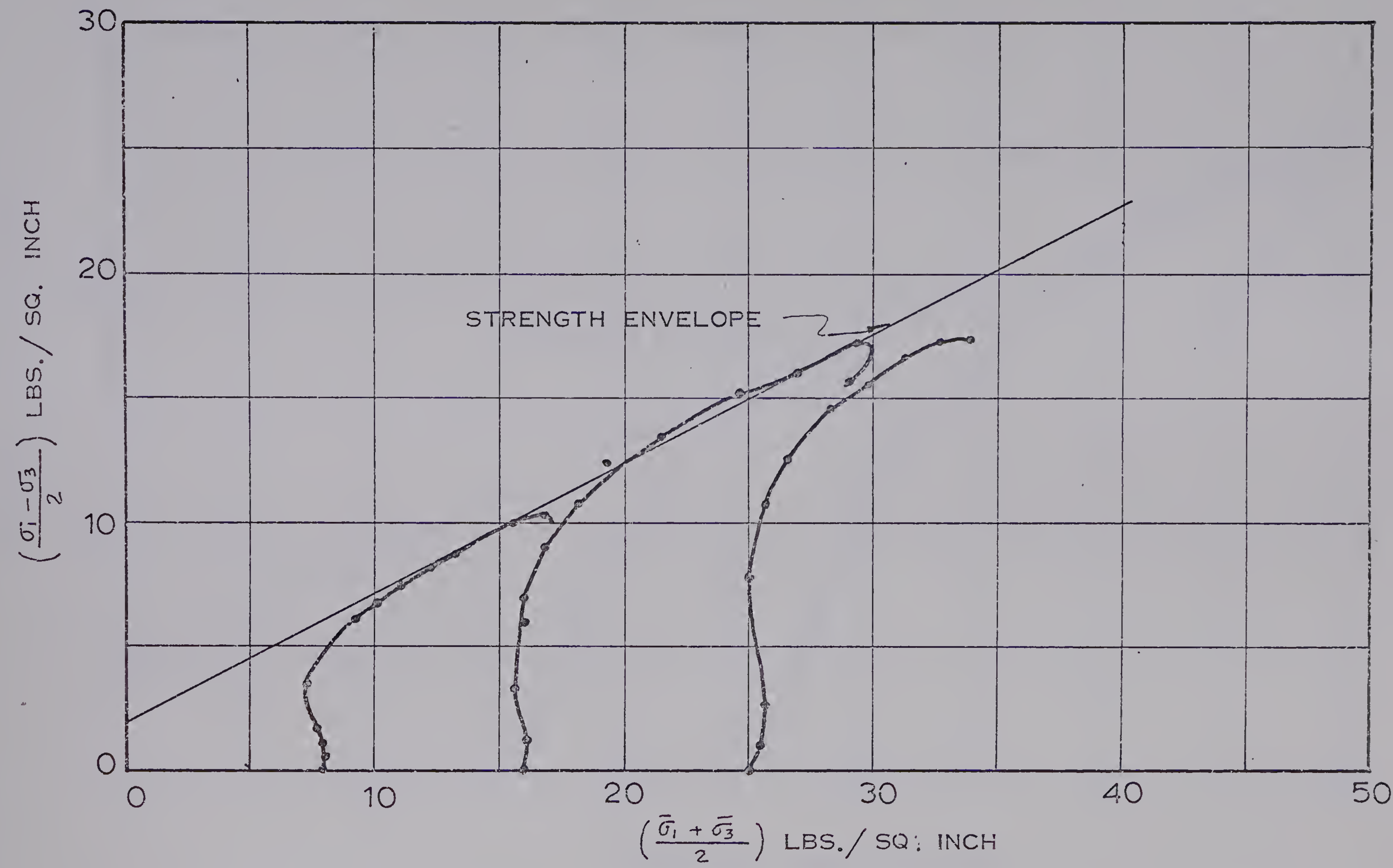



FIGURE V-15. EFFECTIVE STRESS PATHS AND STRENGTH ENVELOPE FOR C-U TESTS ON SAMPLES CBL-I UNDISTURBED

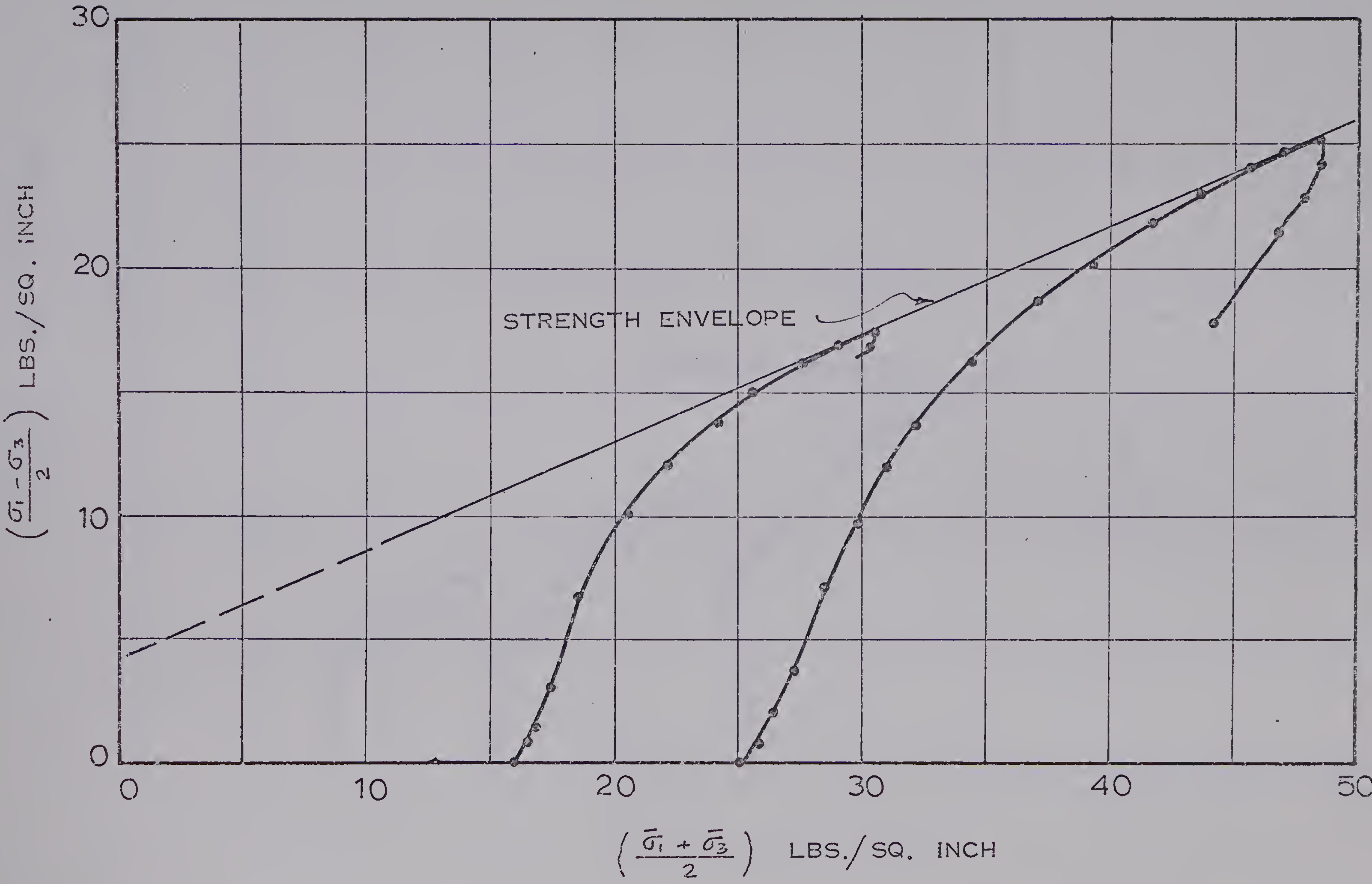



FIGURE V-16. EFFECTIVE STRESS PATHS AND STRENGTH ENVELOPE FOR C-U TESTS ON SAMPLE CBL-1 UNDISTURBED, HORIZONTAL ORIENTATION.

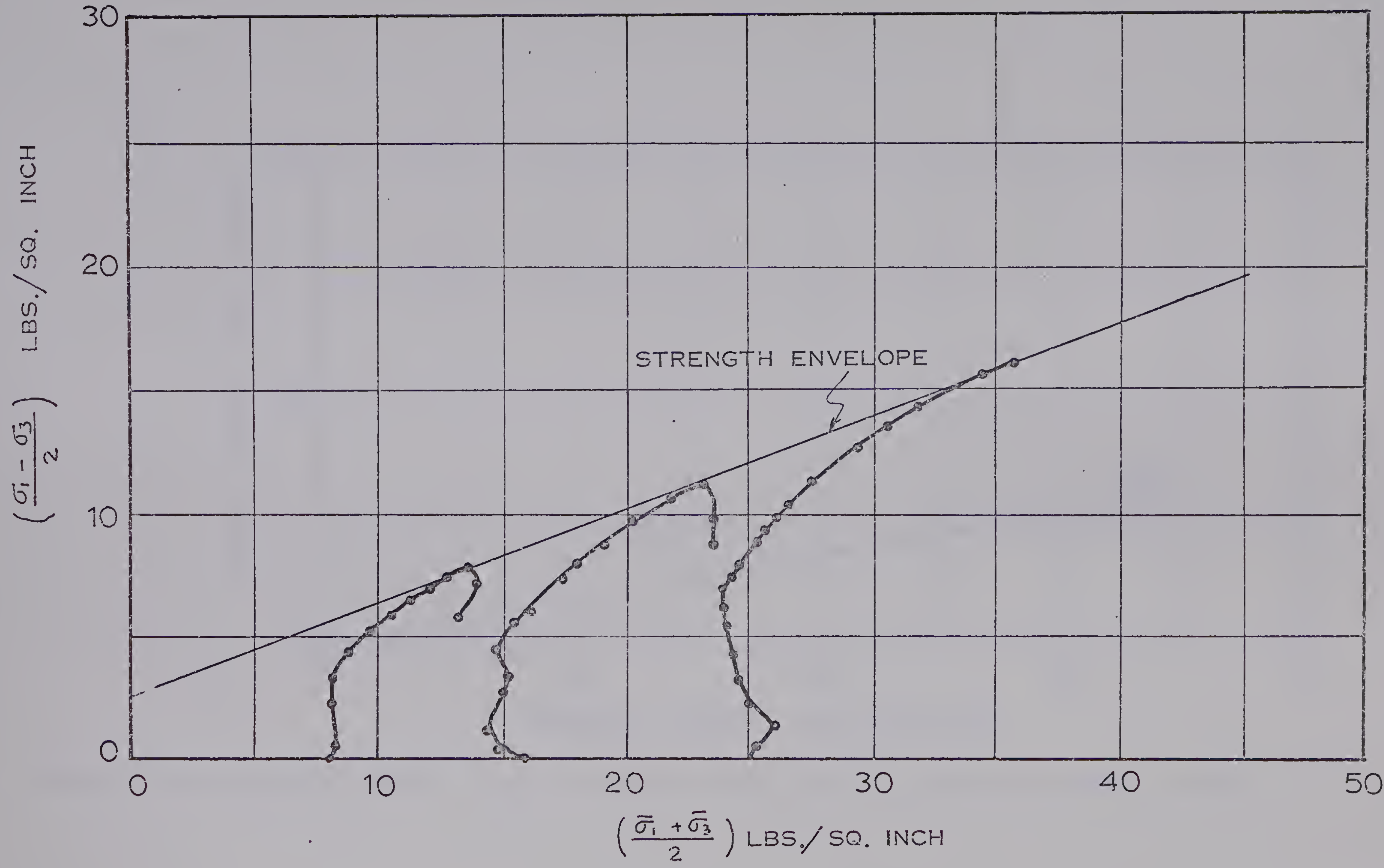



FIGURE V-17. EFFECTIVE STRESS PATHS AND STRENGTH ENVELOPE FOR C-U TESTS ON SAMPLE CBL-3 UNDISTURBED

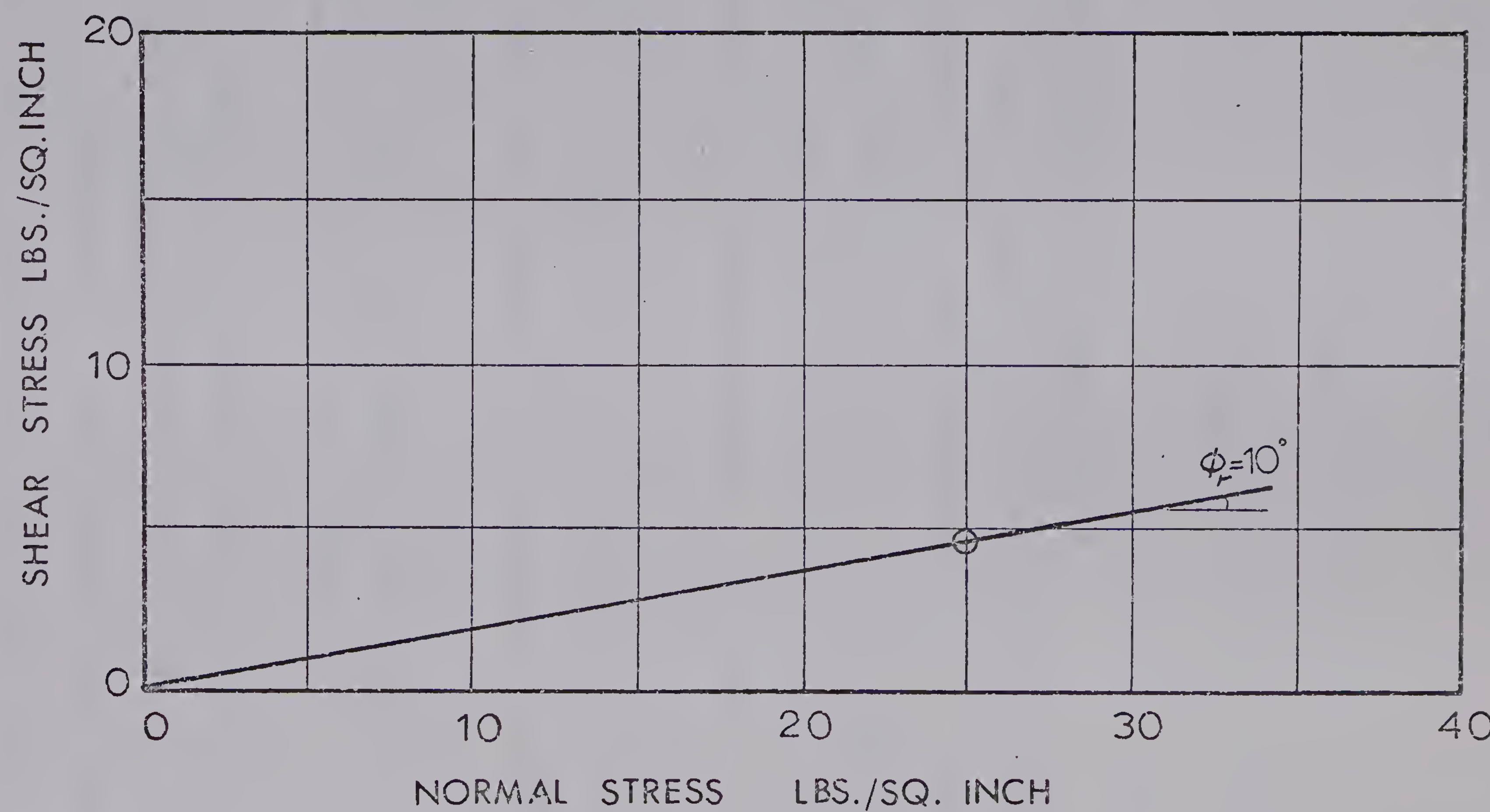



FIGURE V-18. DIRECT SHEAR TEST ON SAMPLE CBL-2 WITH A PRE-CUT SHEAR PLANE.

# CHAPTER VI

# DISCUSSION

## 6.1 Description of the Deposits

From the grain size distribution analyses of the various samples, the major fraction in all soils consists of silt and clay size particles. A very small percentage of gravel is also found associated with all the strata, these may be ice-rafted pebbles. The soils of Project Area C are distinctly stratified. The proportions of the clay sizes in sample CBL-3 is of the order of 84 percent. Samples CBL-2 and CBL-1 from the same area have clay fractions much less than CBL-3. These samples were taken from a lower level in the deposit and suggest that the clay fraction decreases with depth for this deposit.

The soils of Project Area A are siltier and sandier than those of Project Areas B and C. A typical water content diagram for the strata of this area is shown in Figure VI-1. Scattering of the water content from the average value reveals the heterogeneous nature of the deposit.

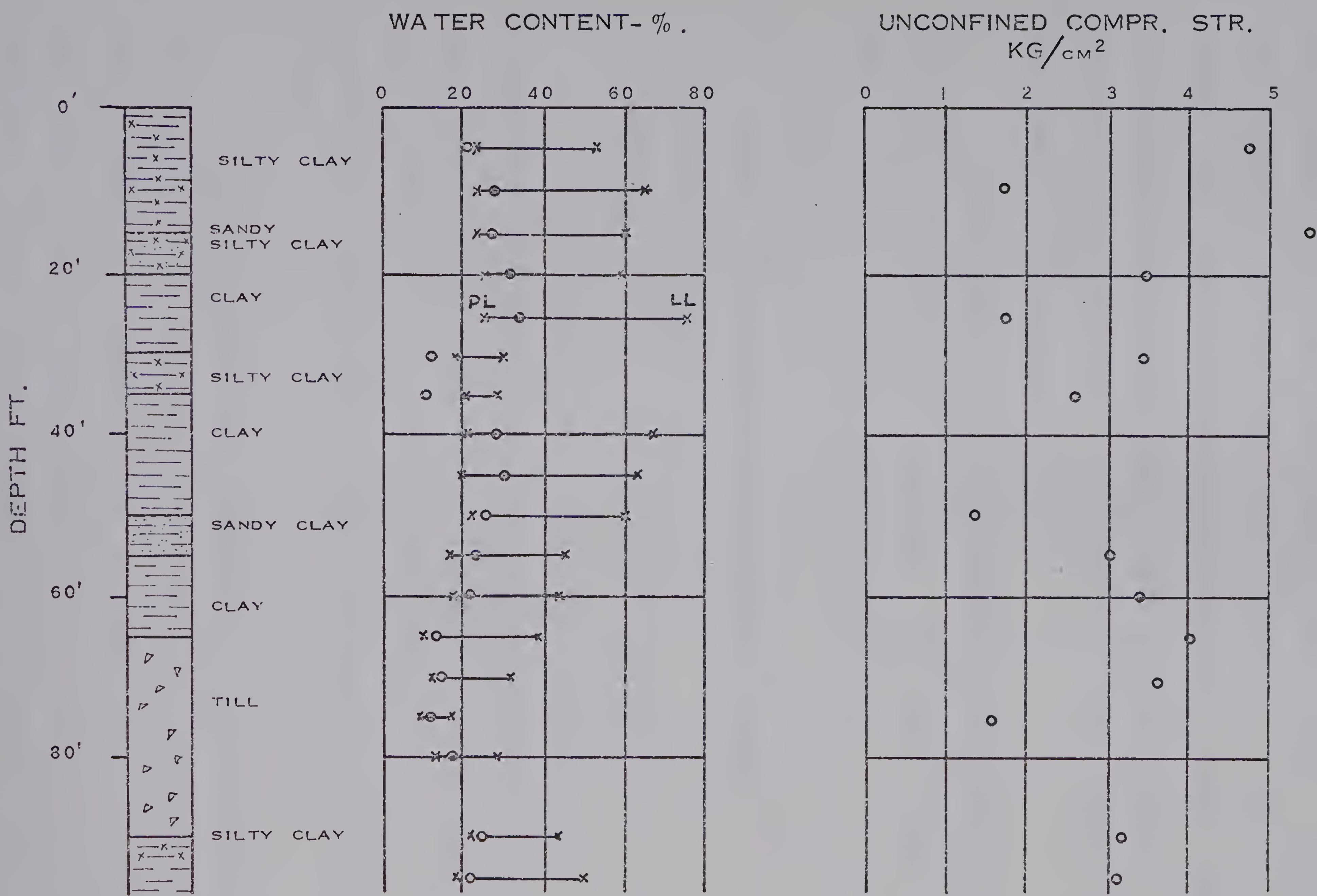



FIGURE VI-1. BORE HOLE NO.1 , UNCONFINED COMPR. TEST RESULTS

[ COURTESY OF DEPT. OF HIGHWAYS, PROVINCE OF ALBERTA ]

The plasticity chart (Figure V-2) shows that the soils tested fall above but close to the A-line. The grouping of points indicates that the soils have a common geologic origin. The range of plasticity values for glacial lake clays suggested by Casagrande (1948), is also shown on Figure V-2 for comparison. The soils of the present study fall close to this range.

Detailed study of the mineralogy of the soils was done by x-ray diffraction analyses. The results indicate that in general the deposits are composed of:

Clay minerals -

Illite, chlorite/kaolinite, montmorillonite.

Non-clay minerals -

Quartz, feldspar, calcite/dolomite, gypsum.

Clay minerals vary from place to place. Soils of Project Area A are rich in illite while montmorillonite is practically absent. The soils of Project Area C show a small percentage of montmorillonite but again illite is predominant. Gypsum crystals are found intercalated between layers.

The activity chart (Figure V-3) shows that the soils tested are transitional between the active and inactive range. This is in spite of the fact that the soils have a clay fraction consisting mainly of illite and not montmorillonite. A possible explanation of this effect could be that

the illites are weathered and there may be a loss of interlayer potassium which would enhance the activity of the illites. There exists a good correlation between the liquid limits and the percent clay fractions of the deposits as shown in Figure VI-2 . Such correlation has been noted elsewhere (Skempton, 1949).

The liquidity indices for the soils are all close to 0.1 which according to Terzaghi (1936) classifies the deposits as stiff clays.

The natural moisture contents of the soils varied from 16 to 35 percent. However, due to exposure on the cut slopes after construction, these moisture contents may not reflect the moisture contents that existed prior to construction.

From the above discussion, based on the classification tests and mineralogy, the material studied can be considered as three relatively distinct material types. These are:

1. Lake deposits - rich in clay fraction, stratified, unified classification CH. (Sample CBL-3).

2. Lake deposits - varved, silty and sandy layers, unified classification CH (Sample CBL-2).

3. Lacustro-till - showing much variation in properties (soils of Project Area A).

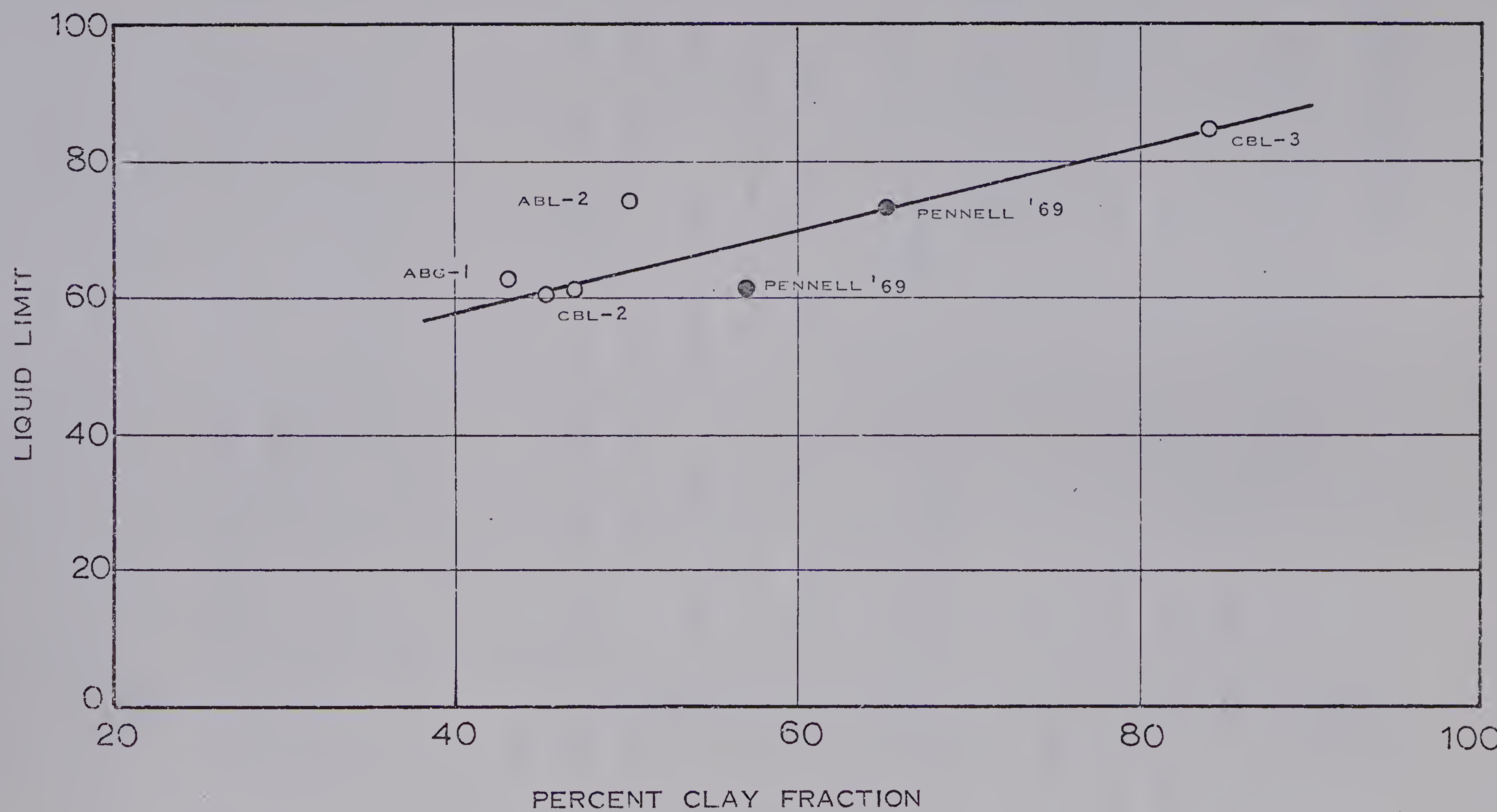



FIGURE VI-2, RELATION BETWEEN LIQUID LIMIT AND % CLAY FRACTION

## 6.2 Shear Strength

### 6.2.1 Stress-Strain Relationships

The majority of the triaxial compression tests performed were consolidated undrained tests with pore pressure measurements. These tests yielded considerable information on the stress strain characteristics of the deposits which is discussed in this section.

These stress-strain relationships may be classified into three types. The samples of Project Area C exhibit a rather steep initial portion of the stress-strain curve. The stress rapidly decreases after attaining peak value at a unit strain of 6 to 7 percent. Sample CBL-3 shows a more pronounced peaking behaviour than CBL-2. In contrast to these two, the samples of Project Area A (e.g. ABL-3) have a curve which is more plastic in nature. For these latter samples, the maximum stress was attained at very large strain and there is no reduction in stress with further strain (within the experimental range). The failure strain of various samples are listed in Table V-4. Photographs of failed specimens of samples from Project Area C are shown in Plate VI.

(a) For sample CBL-3, the peak was reached at 5 to 6 percent strain. The stress-strain curves for this material show a sharp decrease after the peak. The excess pore water pressures are positive before the failure, but

PLATE VI FAILED TRIAXIAL TEST SPECIMENS, SAMPLE CBL-3

become negative as the strain increases (see Figure C-1). These curves indicate a considerable post peak strength decrease. The amount of strain necessary to bring about the sharp decrease in strength is perhaps the most direct indicator of the field performance.

The stress-strain curves suggest that the soil is overconsolidated. High strength and the unstable stress-strain curve (brittle failure) for sample CBL-3 provide evidence of the influence of high density and low liquidity index.

(b) The stress-strain curves for sample CBL-2 show a distinct drop after peak. The drop in strength after peak is 14.3 percent at cell pressure of 25 psi.and 20 percent at cell pressure of 8 psi. The sample CBL-2 is more silty than CBL-3 and appeared less dilatent also.

(c) Figures C-4 to C-8 in Appendix C are the stress-strain curves for the samples of Project Area A. The curves show a considerable variation in behaviour. The curves of test series A on sample ABL-2 show slight peaks. However, the curves of test series B on the same sample show a peak at a cell pressures of 25 psi. The stress-strain curves for samples

of the Project Area A are, in general, smoother in that they do not show a peak and are typical of a glacial till. The stress-strain curves of test series C on samples ABL-2 also indicate a plastic behaviour.

Consolidated drained tests carried out on ABL-2 show distinct peak behaviour. Thus the soil behaviour is more brittle under drained conditions.

The stress-strain curves for sample ABL-3 do not show any peak behaviour which is typical of glacial tills. The excess pore pressures are negative at failure indicating the overconsolidated nature of the deposits which is also revealed by the $A_f$ value as given in Table V-4.

In considering the stress-strain characteristics it is to be noted that the failure zone in case of a brittle failure is very thin and represents only a small percentage of the total volume. The local build up of a pore pressure in a thin zone in the centre of the specimen, as failure is approached would possibly not be recorded at the ends of the sample. Beyond the peak, despite the overall increase in strain, it is anticipated that the actual movement in the failure zone is even more than that measured and that the remainder of the sample rebounds under reducing loads and causes reduction in pore pressures.

### 6.2.2 Failure Envelope and Stress Paths

The stress paths in all the pressure ranges for the tests performed rise to meet the strength envelopes at the point of maximum effective stress ratio and then follow the envelope until failure occurs. From the shape of the stress paths the soils appear overconsolidated. In the following section, each of the three types of soils is discussed.

(a) The failure envelope based on Mohr-Coulomb criterion for sample CBL-3 as shown in Figure V-11 is well defined, giving $\phi = 20^\circ$. The soil exhibits a low cohesion intercept of 2.4 psi. From the Mohr circles the fitting obtained for the failure envelope is excellent, suggesting that the measured strength is representative. The stress paths are typical of overconsolidated soil.

(b) The failure envelope and stress paths for samples CBL-1 (or CBL-2) are shown in Figures V-9 and V-15 respectively. The stress paths again are typical of overconsolidated soils. The cohesion intercept for these soils as well is quite low. To study the strength anisotropy for these samples, triaxial tests were carried out on samples with horizontal orientation* and $\phi'$ was $28.5^\circ$ as against $26^\circ$ for vertical orientation, with zero cohesion. This reveals that material is anisotropic in drained strength. (See Figure V-10).

---

* Horizontal specimen has the cylindrical axis and the major principal stress parallel to the plane of laminations.

(c) For samples of the Project Area A the effective cohesion intercept is close to zero. The effective strength parameters c' and $\phi'$ obtained in C-U and C-D tests for sample ABL-2 are found to be the same thus the strengths measured are felt to be representative.

Tests on remoulded samples do not show any significant decrease in strength and sensitivity can be considered to be practically unity.

From the triaxial compression tests, it is observed that in all the soils there is small or zero cohesion intercept. This is expected of an illitic clay. Other possible explanation of low cohesion is that the stress due to the all-around consolidation pressure breaks some bonds, which in turn causes re-distribution of stress, which overstresses and breaks other bonds. Hence very little cohesion or no cohesion is absorved. Another factor could be that the clay minerals occur as aggregates that are made up of large number of clay particles. In this case the aggregates may be larger than clay size (2μ) and may behave more like silt particles showing no cohesion. Low cohesion could also be probably a consequence of partial saturation, although it may also reflect curvature of the failure envelope. The deposits are, of course partly saturated, either as a result of in-situ conditions above the ground water table, or the drying on the cut slopes. A third suggestion for low cohesion is that there are no bonds present.

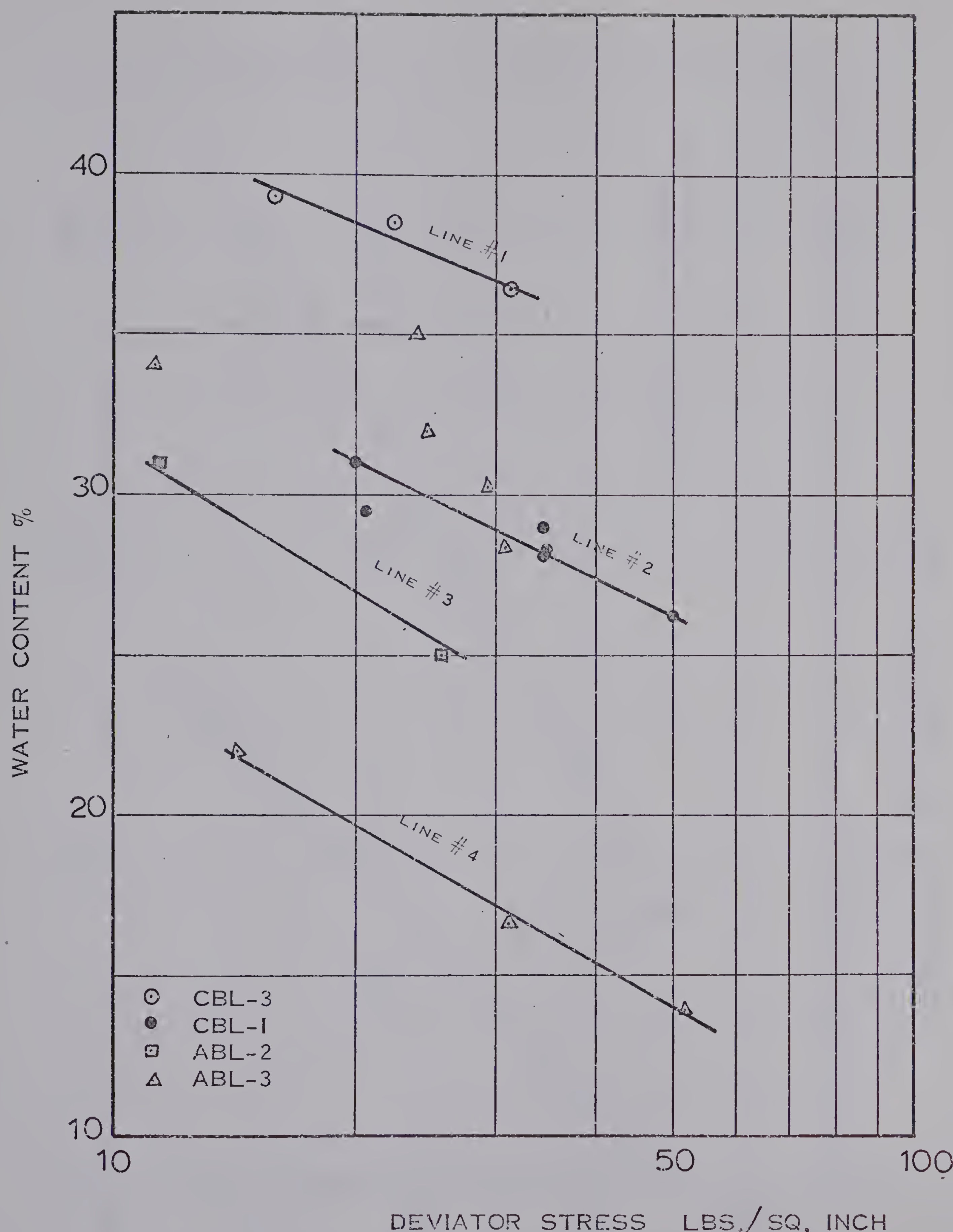



FIGURE VI-3 WATER CONTENT vs LOG STRESS, TRIAXIAL COMPRESSION TESTS

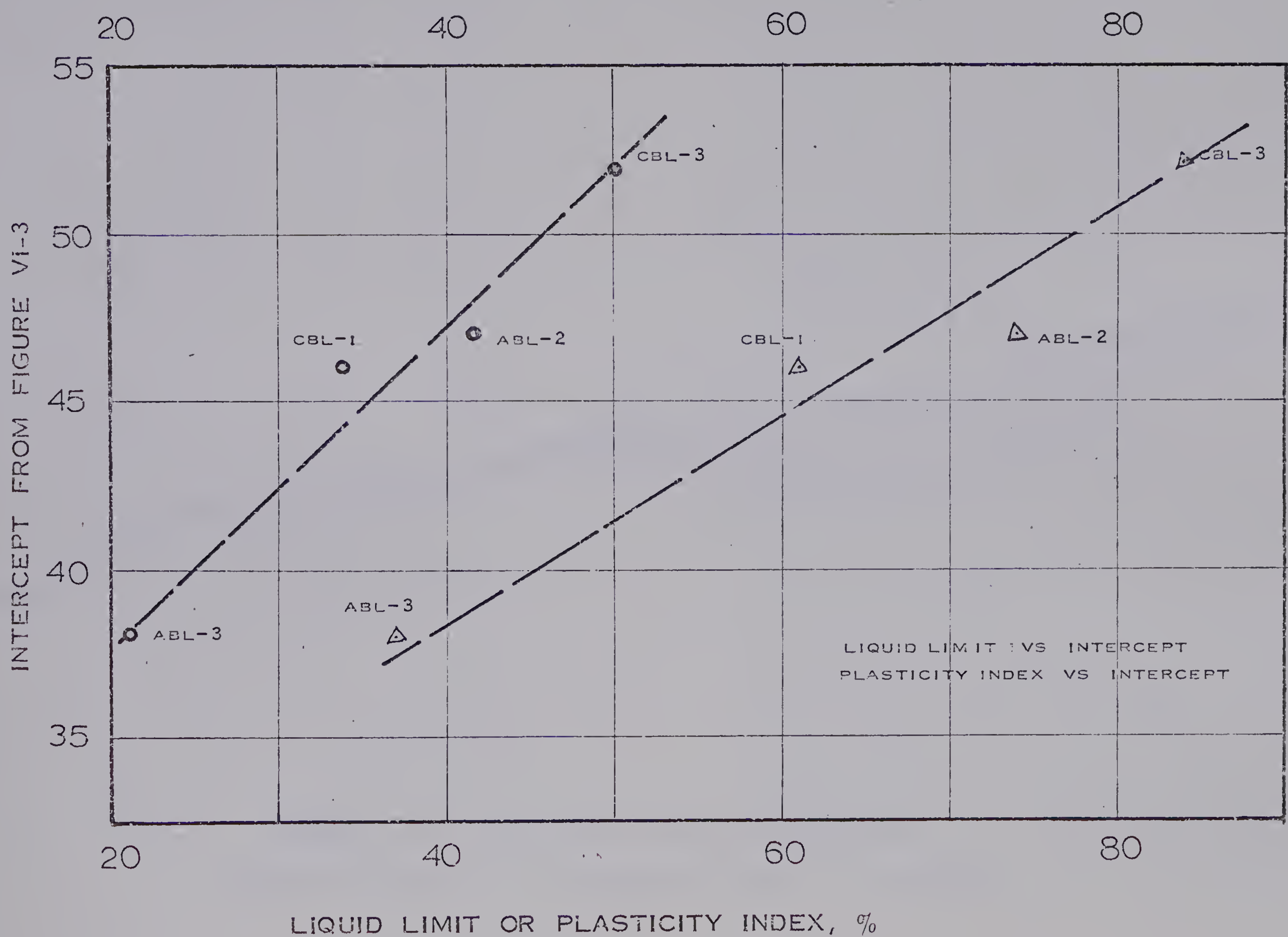



FIGURE VI-4, PLOT SHOWING RELATIONSHIP BETWEEN INTERCEPT FROM FIGURE VI-3 AND LIQUID LIMIT OR PLASTICITY INDEX

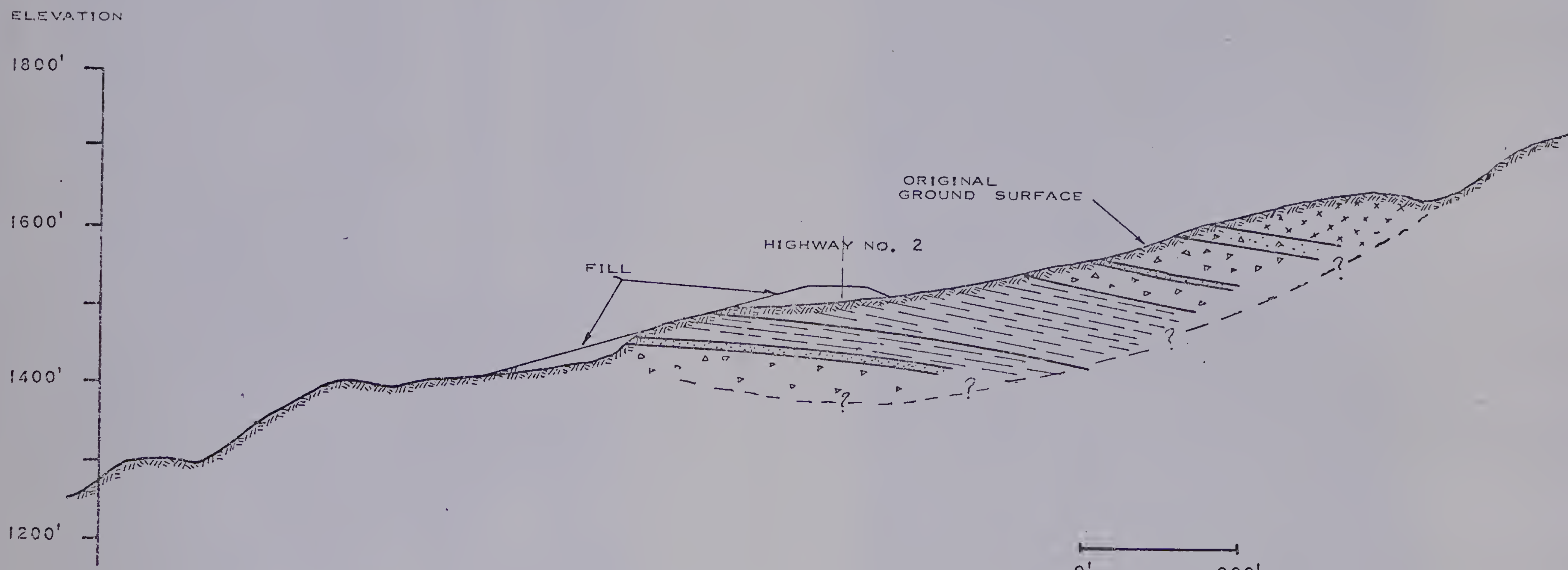



FIGURE VI-5 -SECTION THROUGH SLIDE III
PROJECT AREA 'C', CHAINAGE 1209 + 00 APPROX.

PLATE VII , SHOWING MOVEMENT IN THE FILL
NEAR PROJECT AREA C

A plot of water contents at failure versus the log of maximum deviator stress is shown in Figure VI-3. The plots of Figure VI-3 obtained for each soil are found to be nearly parallel to each other. The equations of these lines are:

Line 1, $w = -2.0 \log_{10}(\sigma_1 - \sigma_3) + 52.2$

Line 2, $w = -2.3 \log_{10}(\sigma_1 - \sigma_3) + 46.2$

Line 3, $w = -2.7 \log_{10}(\sigma_1 - \sigma_3) + 46.9$

Line 4, $w = -2.8 \log_{10}(\sigma_1 - \sigma_3) + 38.3$

In above equations coefficient of $\log_{10}(\sigma_1 - \sigma_3)$ is the slope of the line and the constant is the intercept on the vertical line $(\sigma_1 - \sigma_3) = 1$. It represents the moisture content for a deviator stress of 1 psi.

Within practical limits the slopes of the lines are the same, but the intercepts are considerably different for each soil. A plot of these intercepts versus liquid limits and plasticity indices is shown in Figure VI-4. The intercept increases with increase in liquid limit or plasticity index , which indicates that the strengths of the soils tested are related to the clay fraction. Since it has been shown (Table V-2) that the clay mineral is almost exclusively illite, it would appear that the amount finer than 2μ would also exert direct bearing on the intercept. Unfortunately, the grain size distribution is not available for Sample ABL-3. These data suggest that the fine fraction of all the soils has been locally derived by the glacier from the local bedrock (Shaftesbury Shale).

It is of interest to note that the slopes of the strength lines on Figure VI-3 are parallel despite the fact that the materials arise from different depositional agencies, that is, ABL-3 is a till whereas CBL-3 is a laminated lake deposit. This suggests that the mineralogy and grain size distribution exerts a greater influence on the moisture content-deviator stress relations than does the depositional history. Much more data must be obtained to confirm this suggestion.

### 6.2.3 Residual Strength

A direct shear on a pre-cut plane on Sample CBL-2 revealed $\phi=10°$. A surface profile which involves the same material as CBL-2 (based on drill hole information) is shown in Figure VI-5. The dotted line is a likely slip surface of this failed slope. The $\phi_r=10°$ correlated well with the average slope of this profile (11°). This profile shows pre-construction ground surface, which has now been removed because of a slide (near Project Area C) experienced during the construction of the highway. Visual observation of the site shows that the slope is still not stable. The extent of cracking and the outward movement of the posts for the side railings are evidence movement (see Plate VII).

## 6.3 Consolidation

The consolidation behaviour of soils have been investigated using triaxial and oedometer consolidation tests. In the following section the consolidation characteristics of each soil is discussed.

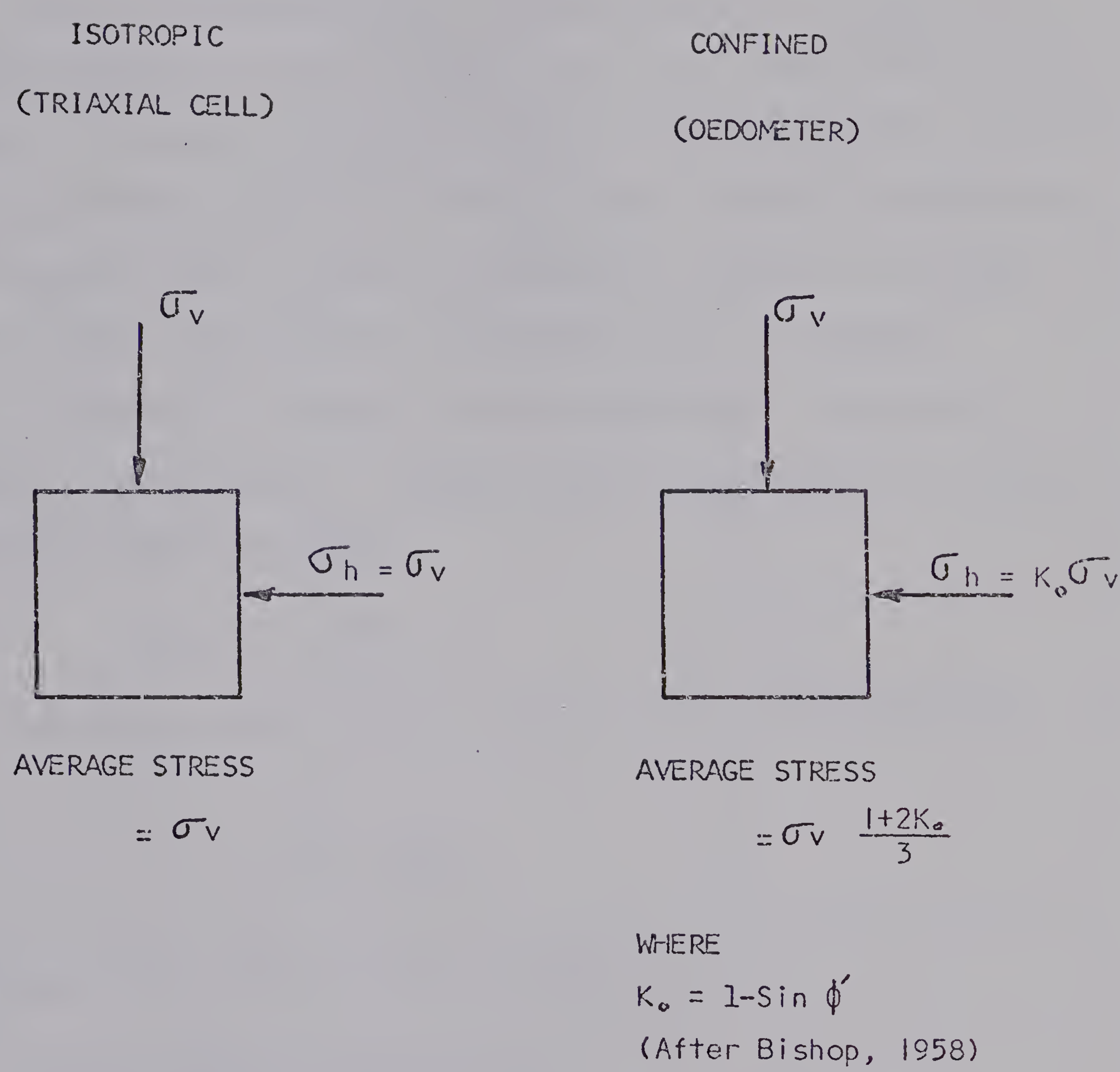



FIGURE VI - 6 ISOTROPIC AND CONFINED COMPRESSION

The various e-log p relationships for the triaxial and oedometer consolidation tests are shown in Figure V-4 and Figure V-5.

In an oedometer test, one-dimensional compression takes place whereas in the triaxial cell the consolidation is isotropic. In order to compare the two tests directly either a $K_o$ consolidation in the triaxial cell should be performed or the oedometer test results presented in terms of average stresses rather than vertical pressures only. Consider elements in Figure VI-6 under uniaxial confined compression and triaxial compression. Bishop (1958) presented the following empirical relationships:

$$K_o = 1 - \sin \phi'$$

Average stress in an oedometer can be represented as:

$$= \sigma_v \frac{(1 + 2K_o)}{3}$$

and Average stress in a triaxial cell $= \bar{\sigma}_c$ .

In the above expressions:

$\sigma_v$ = Effective vertical stress in oedometer

$\bar{\sigma}_c$ = Effective cell pressure in triaxial consolidation

$K_o$ = Ratio of vertical to horizontal stresses in one-dimensional consolidation

$\phi'$ = Effective angle of internal friction.

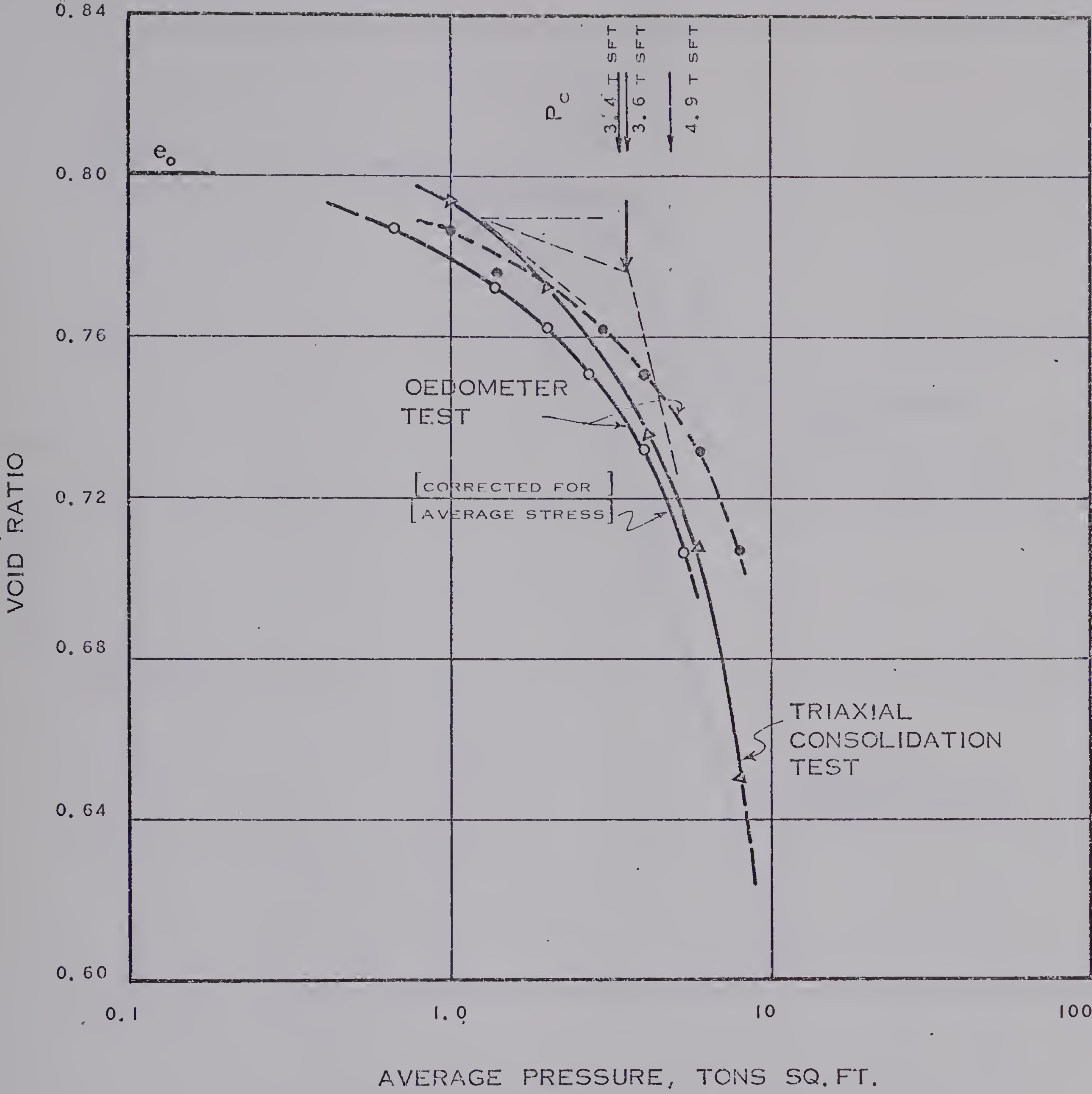



FIGURE VI-7, OEDOMETER AND TRIAXIAL CONSOLIDATION
SAMPLE CBL-2

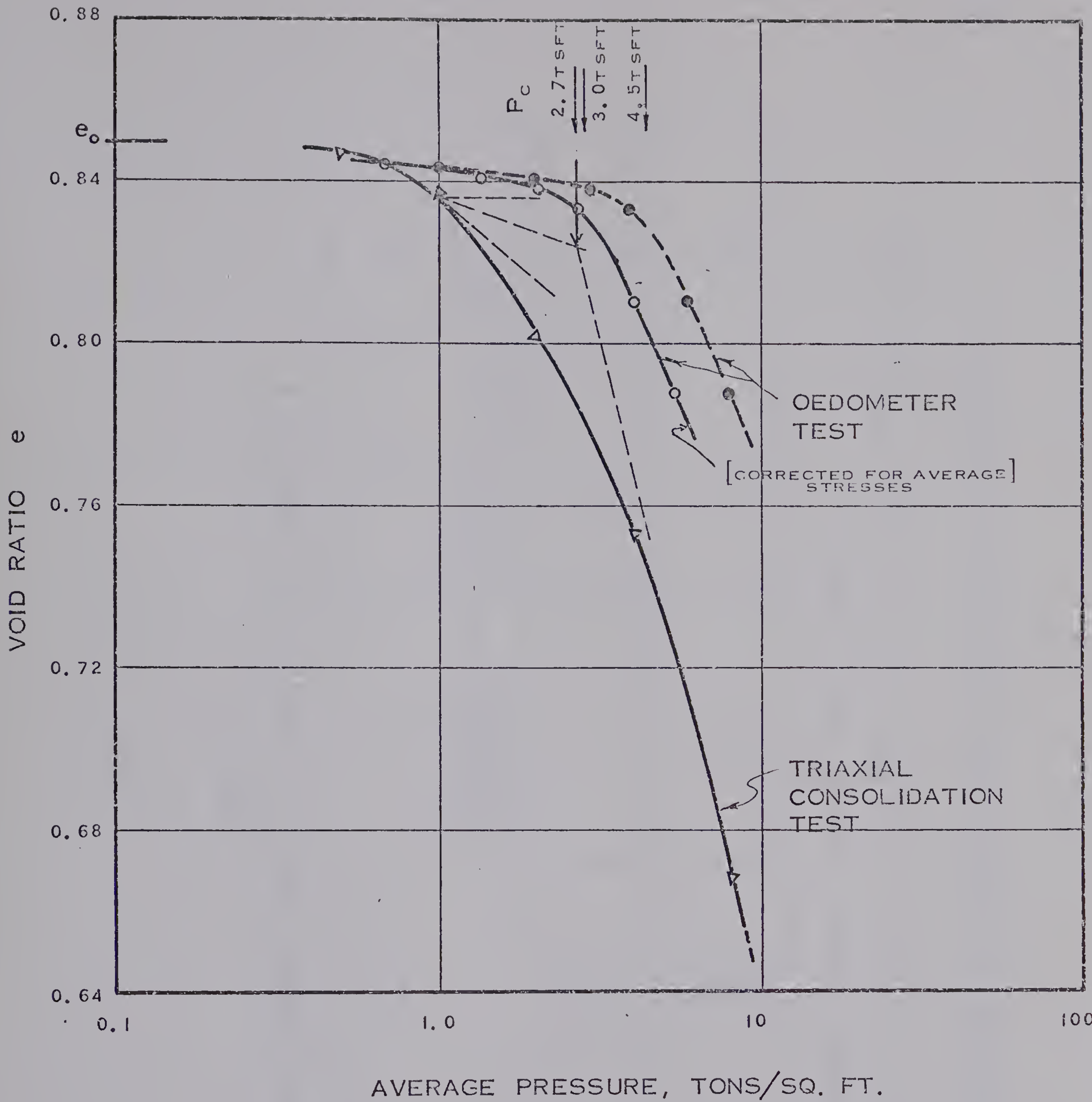



FIGURE VI-8, OEDOMETER AND TRIAXIAL CONSOLIDATION
SAMPLE CBL-3

TABLE VI-1

PRE-CONSOLIDATION PRESSURES, SAMPLES CBL-2 AND CBL-3.

| Sample | $\bar{p}_o$ | $\bar{p}_c$ Oedometer one-dimensional compression | $\bar{p}_c$ Oedometer corrected for average stress | $\bar{p}_c$ Triaxial Cell |
|---|---|---|---|---|
| CBL-3 | $2.4_{(1)}$* <br> $4.8_{(2)}$ | 4.5 | 3.0 | 2.7 |
| CBL-2 | $2.9_{(1)}$ <br> $5.7_{(2)}$ | 4.9 | 3.4 | 3.6 |

$\bar{p}_o$ = present overburden (before construction), t/sft.

$\bar{p}_c$ = pre-consolidation pressure calculated by Casagrande's method, t/sft.

* (1) -based on submerged unit weights for the soil mass of 60 lb/cft.
(2) -based on total unit weights for the soil mass of 120 lb/cft.

From Figures VI-7 and VI-8 the pre-consolidation pressures, determined by Casagrande's graphical technique, are given in Table VI-1.

From the Table VI-1, oedometer tests show (based on total unit weights) the soils to be overconsolidated with an overconsolidation ratio of about 2, whereas triaxial consolidation tests (based on submerged unit weights) show that soils are normally consolidated or lightly overconsolidated.

Using Casagrande's technique the pre-consolidation pressure, $P_c$, which is associated with the interpretation of consolidation test, is a reflection of the maximum vertical stress that the soil has experienced in its geologic history. Kenney (1968) suggests that this can certainly be the case, but other factors may affect the pre-consolidation pressure. For example, any change in the soil that will cause increased shear resistance between individual soil particles will cause increased resistance of the soil to volume compression, and hence will cause an increase of $P_c$ that is quite independent of applied stress. Increased value of $P_c$ could also be shown because of desiccation. Thus there are many uncertainties in $P_c$ exhibited by a soil in a consolidation test. A pre-consolidation pressure, $P_c$, is dependent on the geologic history of the soil (both stress and environmental histories) rather than only the maximum applied stress it has experienced during its history (Kenney, 1968).

The soils of sample CBL-3 and CBL-2 are lake sediments, therefore, the water table was quite high prior to much erosion by the river. As the river valleys deepened the water table was lowered considerably and soil moisture contents were lowered due to this ground water lowering. The higher pre-consolidation pressures obtained from oedometer tests is evidence of this. However, triaxial consolidation corresponds to the present overburden (pre-construction). This assumes that the present overburden is the same as any previous overburden. The geology of the area, however, suggests that the deposits may have been subjected to glacial pressures, and if so, should be overconsolidated. The consolidation results of this thesis thus lead to the following anomalies:

(a) The gradual lowering of ground water appears to be a triaxial consolidation process. But the results do not agree with the triaxial consolidation in the laboratory. Therefore, lowering of water table in a strata corresponds to one-dimensional loading.

(b) The deposits should have consolidated under the present overburden. The samples of Project Area A, however, do not correspond to present overburden. The points which remain to be explored in this regard are:

- is consolidation still going on ?
- dense structure inhibits consolidation after lowering of ground water table.
- expulsion of air may not be revealed in oedometer test as as over-consolidation effect.

Intactness of the deposits is questionable again. It is hard to establish whether considerable erosion has taken place or the deposits are debris of old landslides.

Lowering of ground water table has certainly influenced the consolidation characteristics of the deposits, as indicated by the results of this thesis.

It is interesting to note that the deposits do not show consolidation due to the ice pressures. It may be possible that the upper unassorted till deposits were deposited as turbidites and never were overridden by the ice. Another possibility is that the pore pressures generated by the ice may not have fully dissipated, hence there was no increase in effective stresses.

# CHAPTER VII

# CONCLUSIONS AND RECOMMENDATIONS

## 7.1 Conclusions

Based on classification tests, mineralogy and strength tests, the following three types of soils have been studied in the present investigation.

(a) Glacial lake deposits - varved clay with silt and sandy layers.

(b) Glacial lake deposit - stratified, rich in clay sizes

(c) Glacio-lacustrine till - showing a wide range of properties and heterogeneous in nature.

The deposits are of glacial origin and the plasticity characteristics correlate with other types of glacial deposits as reported by Casagrande (1949).

The clay mineralogy of the deposits is predominantly illite. The activity chart shows that deposits are of similar geologic origin and derived from parent material rich in illite.

From the information based on field investigation and available drill hole data the deposits are erratic. The presence of cohesionless sand partings and sand pockets are evident and, in this instance, form the planes of weakness in the strata. Because of the general dip of the deposits toward the river, the thin sand layers in the fine grained sediments could become aquifers. The uplift pressures beneath the toe of the natural slopes would reduce the effective stresses in these layers. It is possible, therefore, that the stability of the slopes in the area is decreased by the obvious ground water flow.

The laboratory shear tests show that the deposits of the study area posses high peak strength. However, the strength decreases considerably at strains greater than that for the peak stress. The existing slopes are susceptible to further land slides due to the presence of old land slides, since the shear strength on the slip surfaces is at the residual angle of shearing resistance.

The soils investigated are insensitive to remoulding, i.e. sensitivity is practically unity.

The cohesion intercept is very low for the deposits.

Geological sections reveal a complex Pleistocene history. Varved lacustrine sediments are overlain by sand, tills and silt.

The consolidation results are quite complex and no definite assissment of the geologic history can be made because the data available is too localized. The samples for the present series of tests were collected from the suface of the cuttings and have apparently been subjected to drying. Consequently the liquidity indices are low. Samples being taken from a shallow depth appear to have desiccated.

## 7.2 Recommendations

To be able to make an assessment of the geologic history, in particular the effects of ground water lowering, further research should be directed to study the consolidation characteristics of the soils.

The soils being heterogeneous, careful techniques for testing and design should be applied. In the case of the laminated lake clays of Project Area C, study of individual layers should be made in order to understand the deposits fully.

To study the stability of existing slopes a thorough ground water investigation should be made and piezometers. installed to assess the pore pressures.

Average valley slopes correspond to the residual angle of shearing resistance. Further research in residual strengths may contribute to the understanding of the stability

strengths may contribute to the understanding of the stability of slopes in the area.

An effort may be made to study if solifluction is going on due to the evident ground water movement towards the river valley.

A knowledge of the Pleistocene Geology of the area would be of considerable value to geotechnical engineers as it is likely that similar deposits occur in Northern Alberta.

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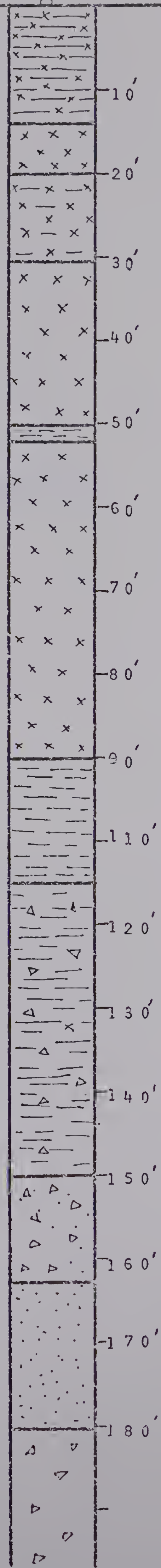
Appendix A

TYPICAL DATA SHEET

FOR CUT-SLOPE LOGS

# LOG OF CUT-SLOPES

Project Area A Chainage 1175+00 Bearing N 10° W




| Description | Legend | Distance | Remarks |
|---|---|---|---|
| Silty clay, light grey fissured, wet stratified | | | glacial lake sediments |
| Silt, light colour | | | |
| Silty clay, fissured, highly plastic, wet | | | |
| Silt, light colour, dry | | | |
| Clay layer, light grey | | | Dip 26° N60 E |
| Silt, light colour, dry with few wet spots | | | |
| Clay, dark grey, very soft, fissured, slickensides, | | | Dip 30° N50° E |
| Clay, with gravels sizes fine to ¼ , soft | | | ice-rafted pebbles |
| Sandy till, from fine to boulder sizes | | | |
| Clean sand | | | Outwash sand |
| Till, dark grey, clayey plastic | | | |

# Appendix B

# TRIAXIAL CELL INSTRUMENTATION

# INSTUMENTATION

The triaxial tests were carried out with a standard cell for 1-$^1/_2$ inches diameter samples and Bishop's self-compensating mercury control system for controlling the cell pressures.

Deviator stresses were measured with a load cell, which was made from a thin wall hollow aluminum cylinder to which were attached strain gauges such that axial and circumferential strain gauges were on alternate arms of a Wheatstone bridge circuit. As long as the cylinder remains within the elastic limits a linear relationship is obtained between the load and output from strain gauge bridge circuit.

Vertical displacements were measured with a linear variable differential transformer (LVDT). This consists of a core placed between two identical secondary windings. When the core is displaced different voltages are induced in the two coils and a signal equal to the difference of these voltages is obtained. The output is proportional to the displacement of the core over a wide range. The circuit diagram in Figure B-1 shows the principle of an LVDT.

Figure B-2 gives the arrangement of the triaxial

cell stilizing the load cell, LVDT, and a pressure transducer. Plate B-1 shows the set up of the triaxial cell and the digital voltage recorder employed to read the output from the instrumentation set up.

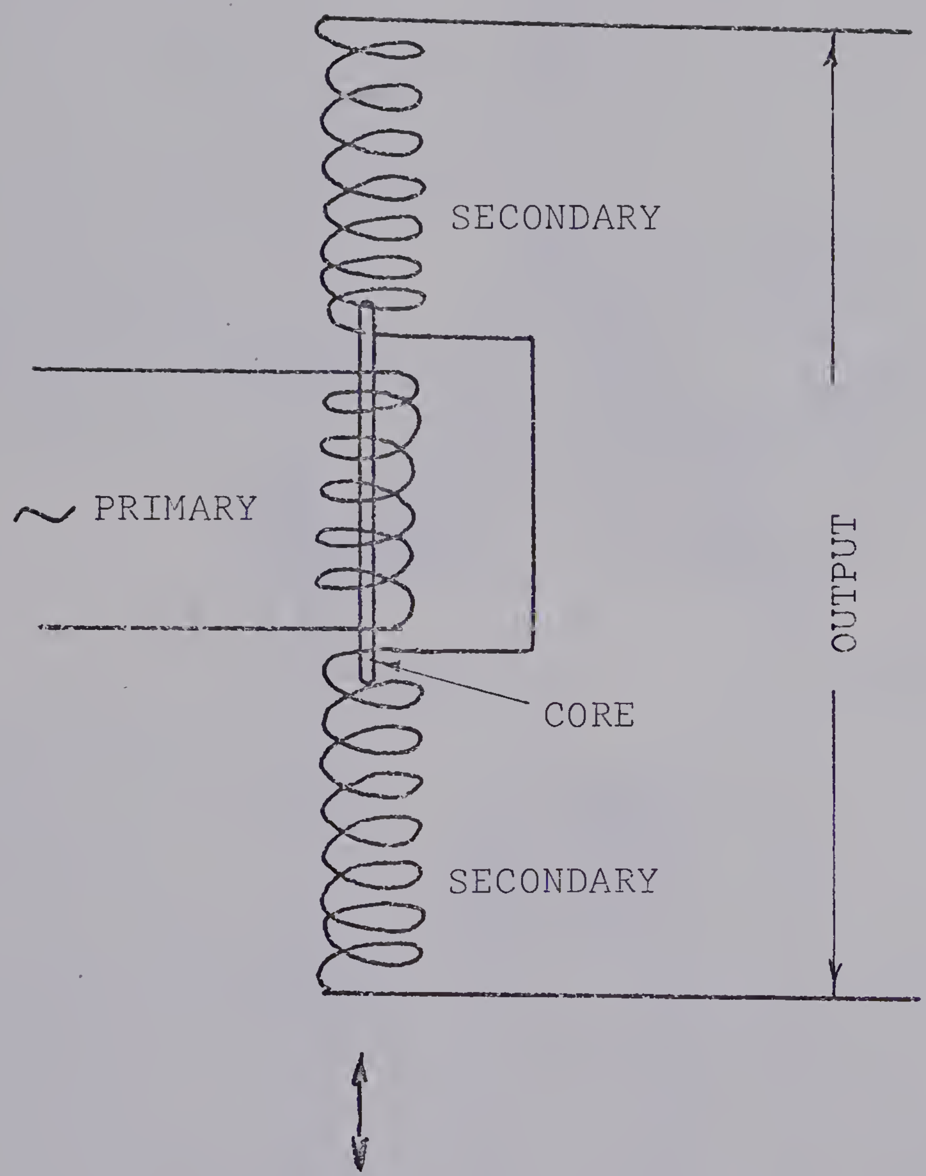



FIGURE B-1  LINEAR VARIABLE DIFFERENTIAL TRANSFORMER

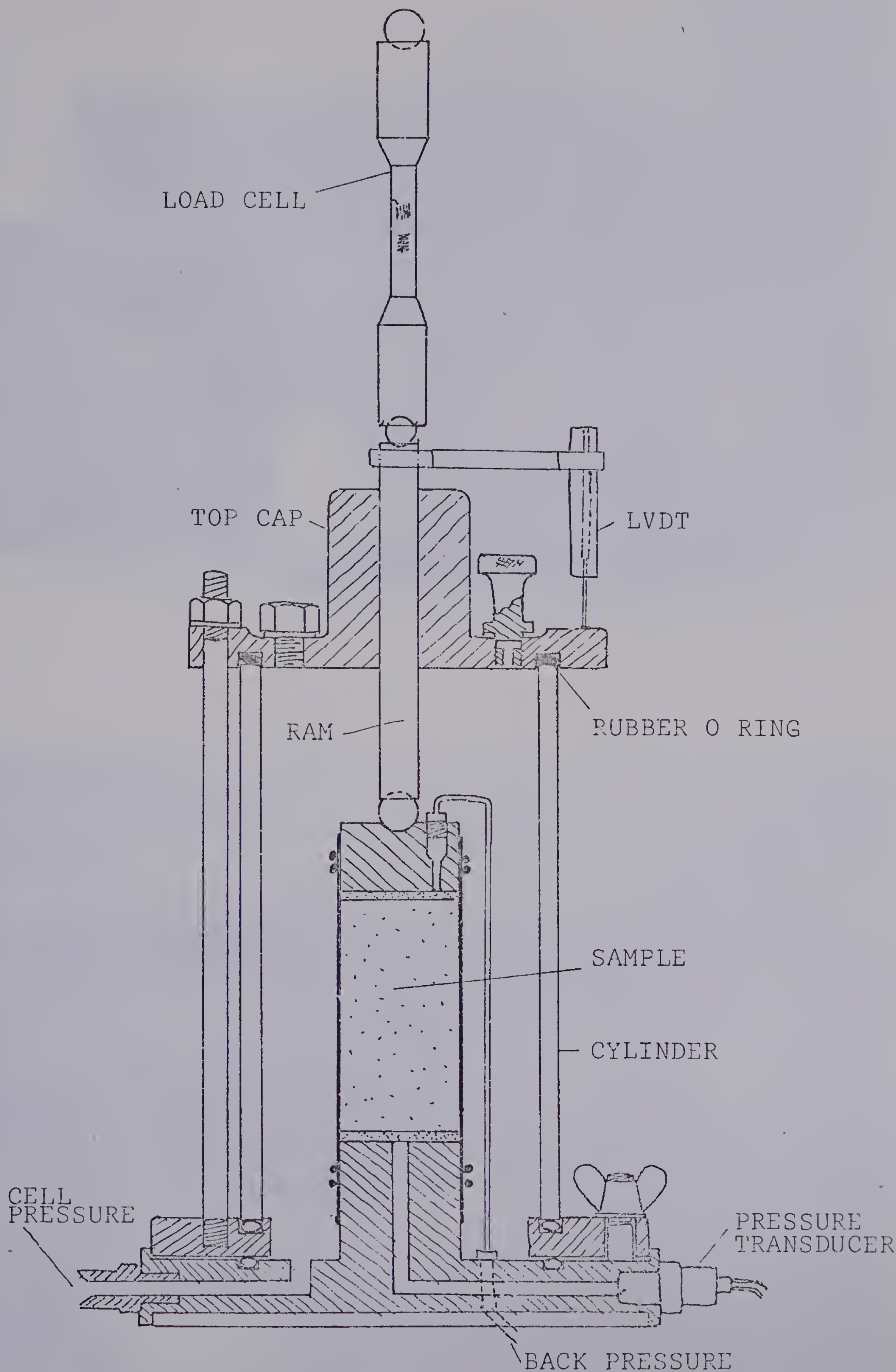
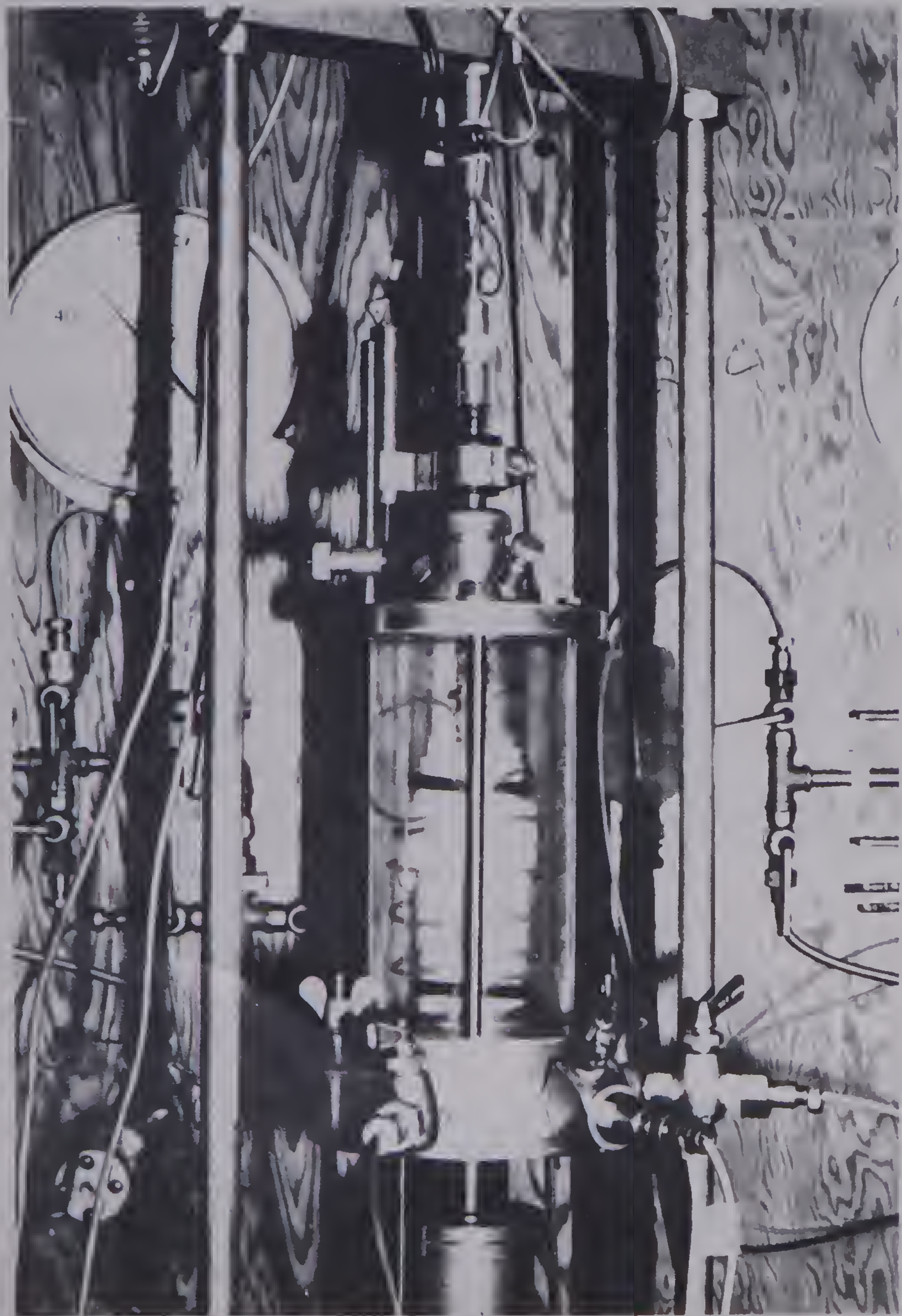
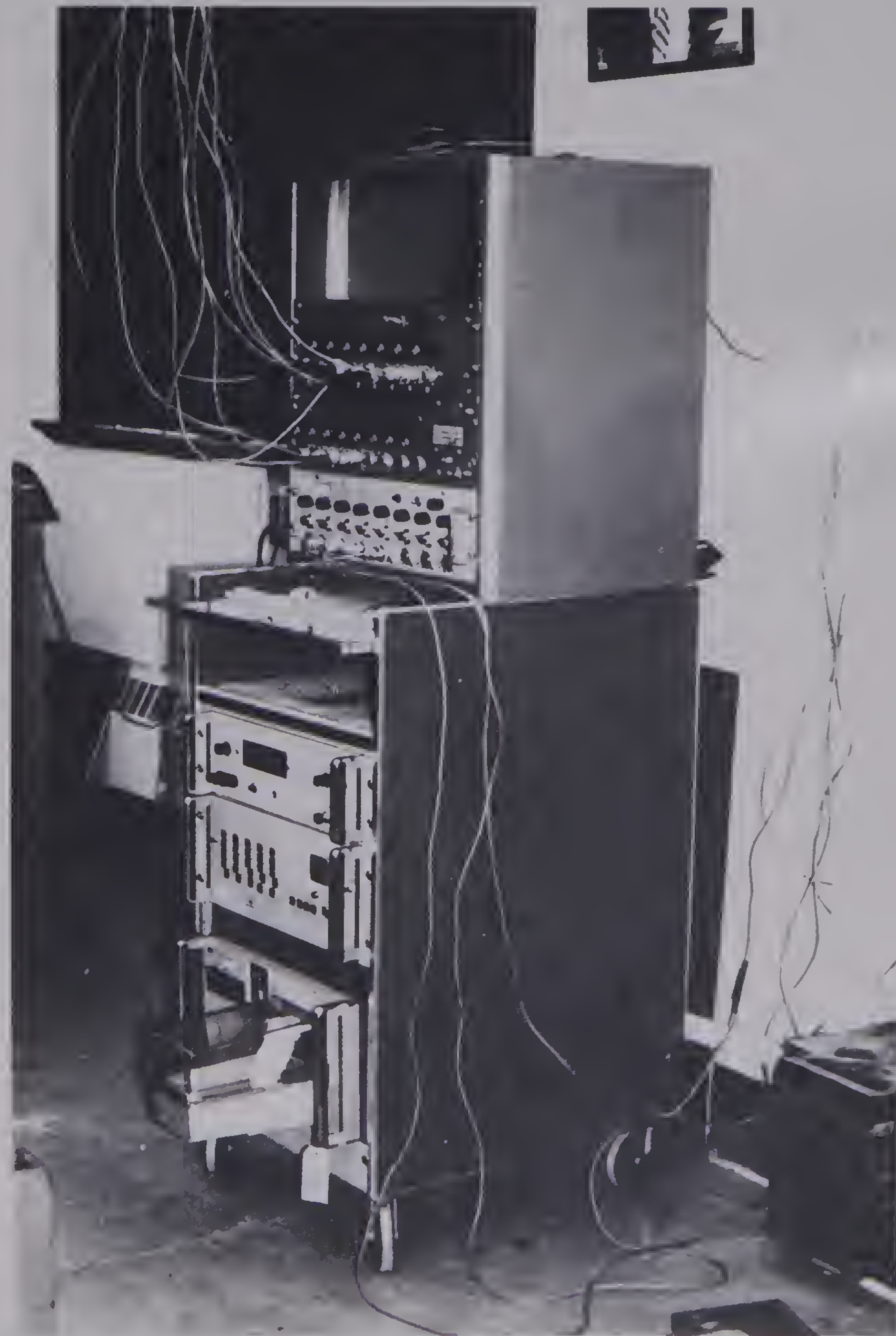



FIGURE B-2 TRIAXIAL CELL ARRANGEMENT AND INSTRUMENTATION



TRIAXIAL CELL



DIGITAL VOLTAGE RECORDER

PLATE B-1

# Appendix C

# STRESS-STRAIN RELATIONSHIPS

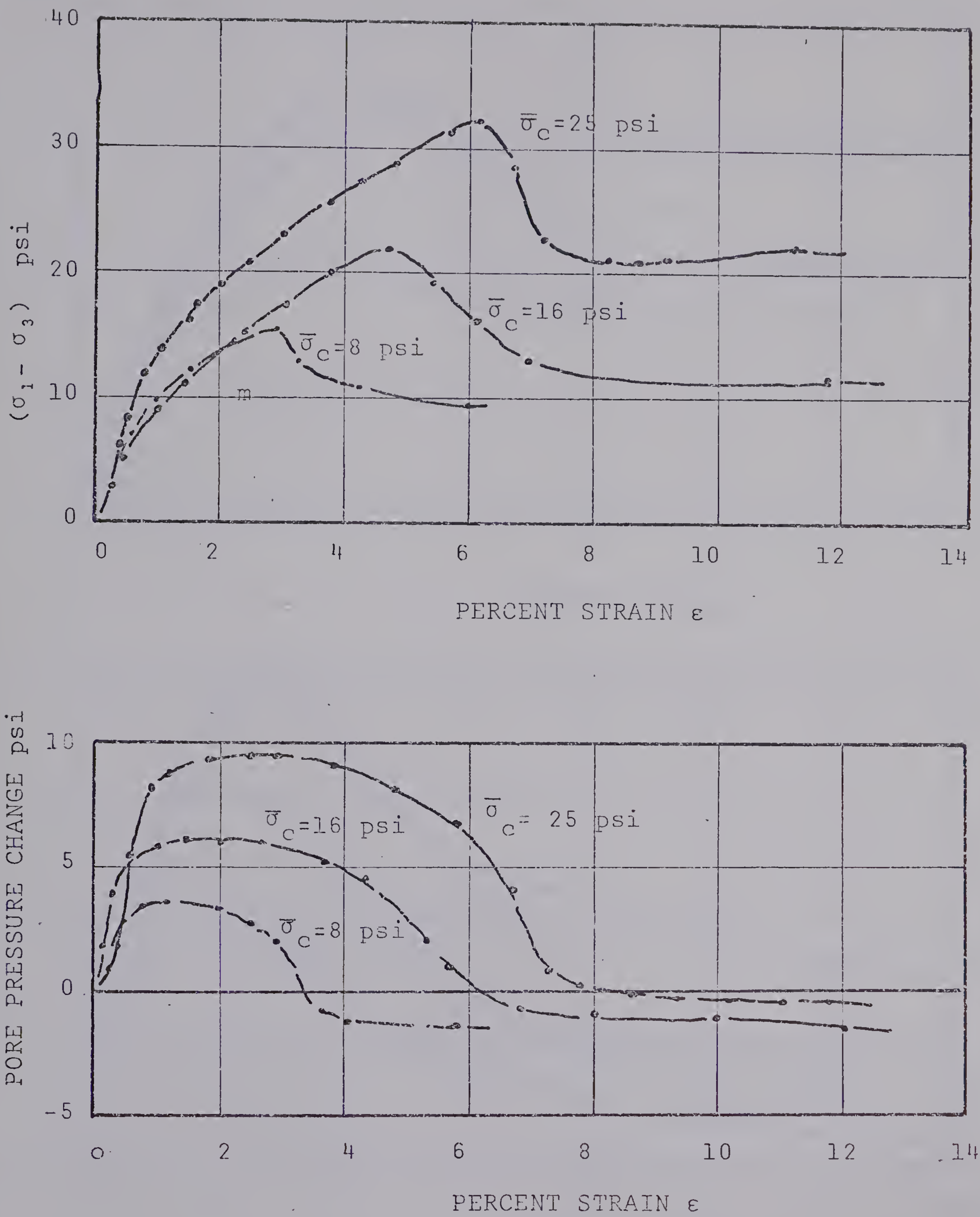



FIGURE C-1 STRESS STRAIN RELATIONSHIP, SAMPLE CBL-3
C-U UNDISTURBED

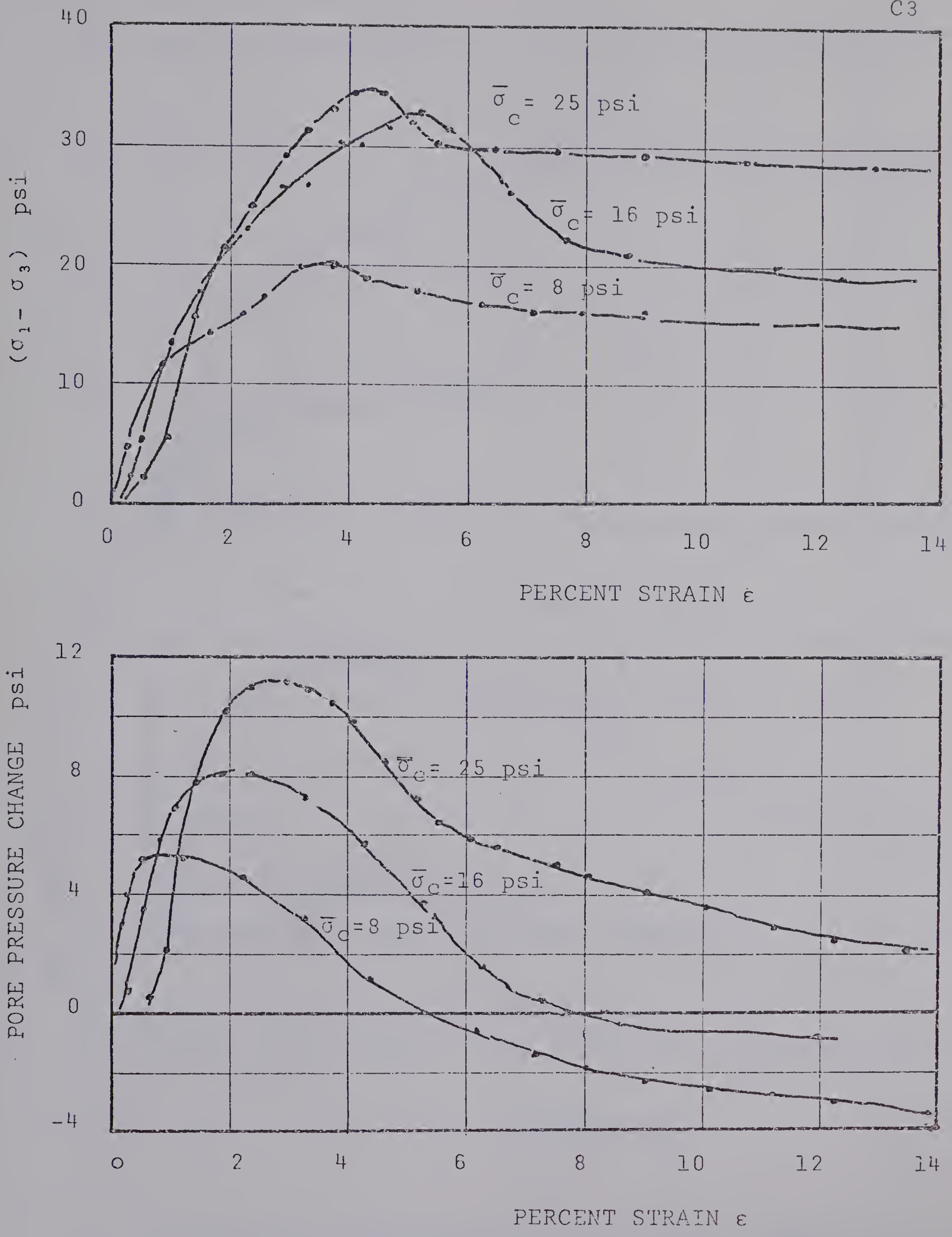



FIGURE C-2 STRESS-STRAIN RELATIONSHIP, SAMPLE CBL-1/CBL-2
C-U UNDISTURBED

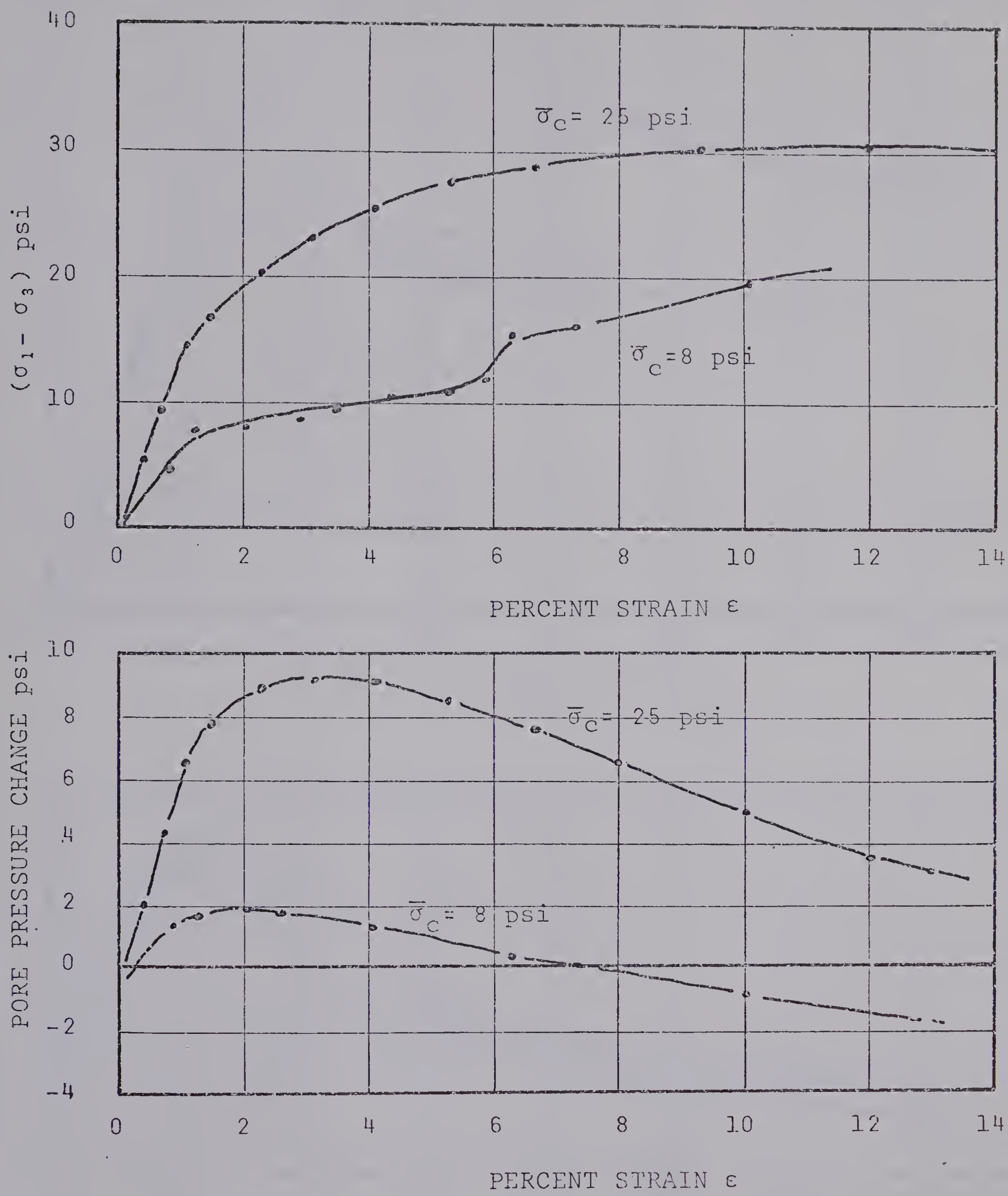



FIGURE C-3 STRESS-STRAIN RELATIONSHIP, SAMPLE CBL-1
C-U REMOULDED

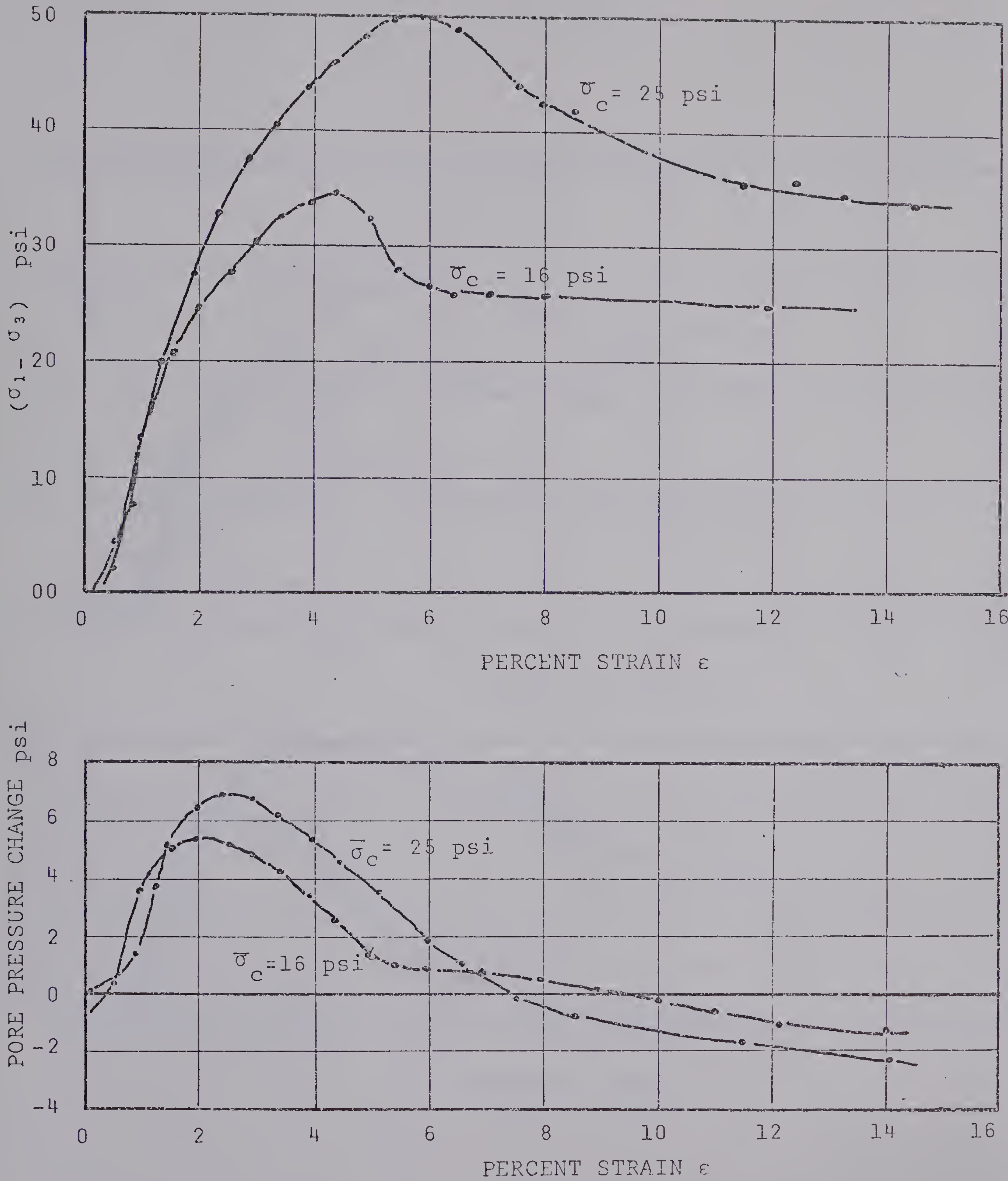



FIGURE C-4 STRESS-STRAIN RELATIONSHIP, SAMPLE CBL-1
C-U UNDISTURBED

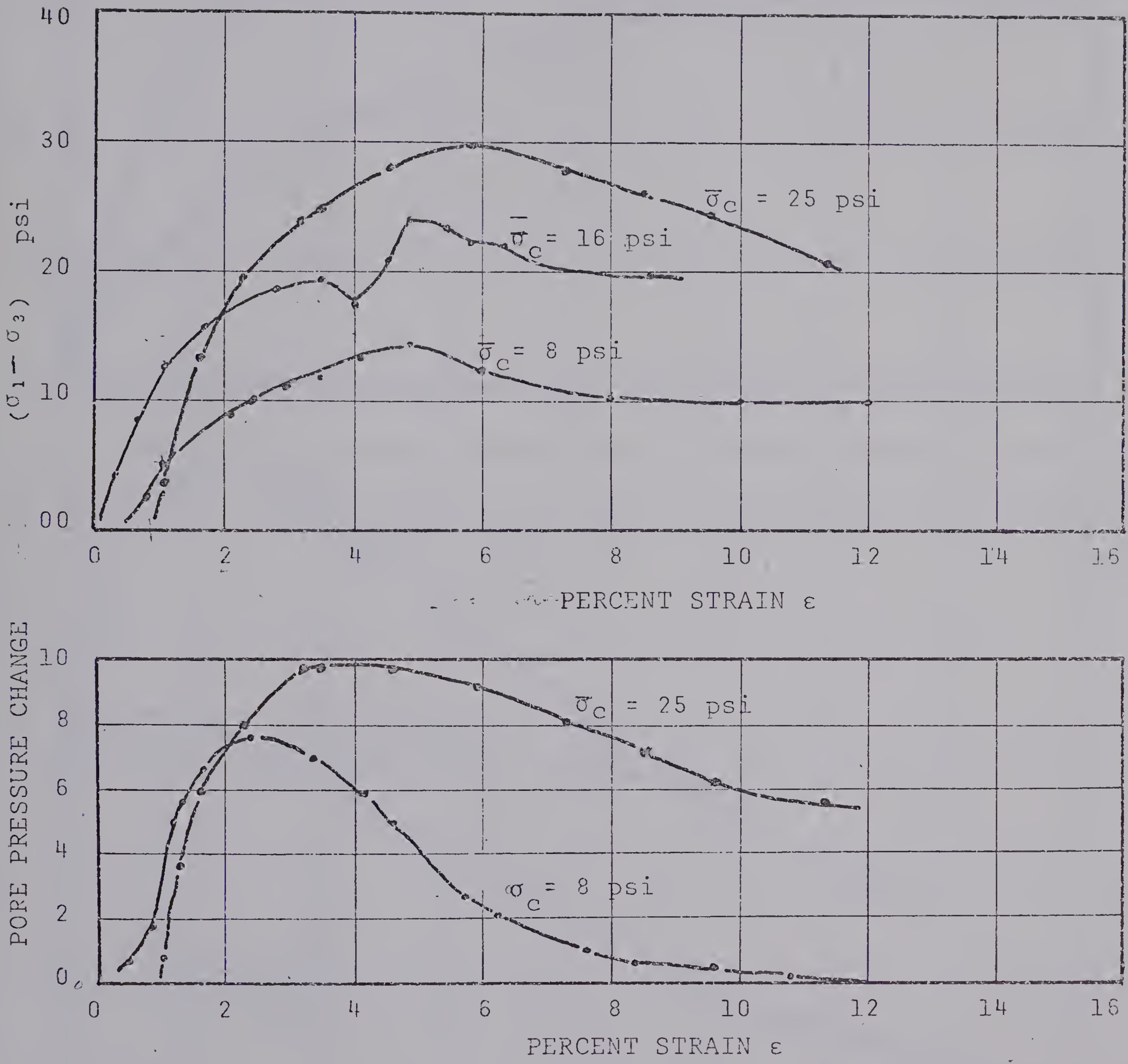



FIGURE C-5 STRESS-STRAIN RELATIONSHIP, SAMPLE ABL-2
C-U UNDISTURBED, TEST SERIES 'A'

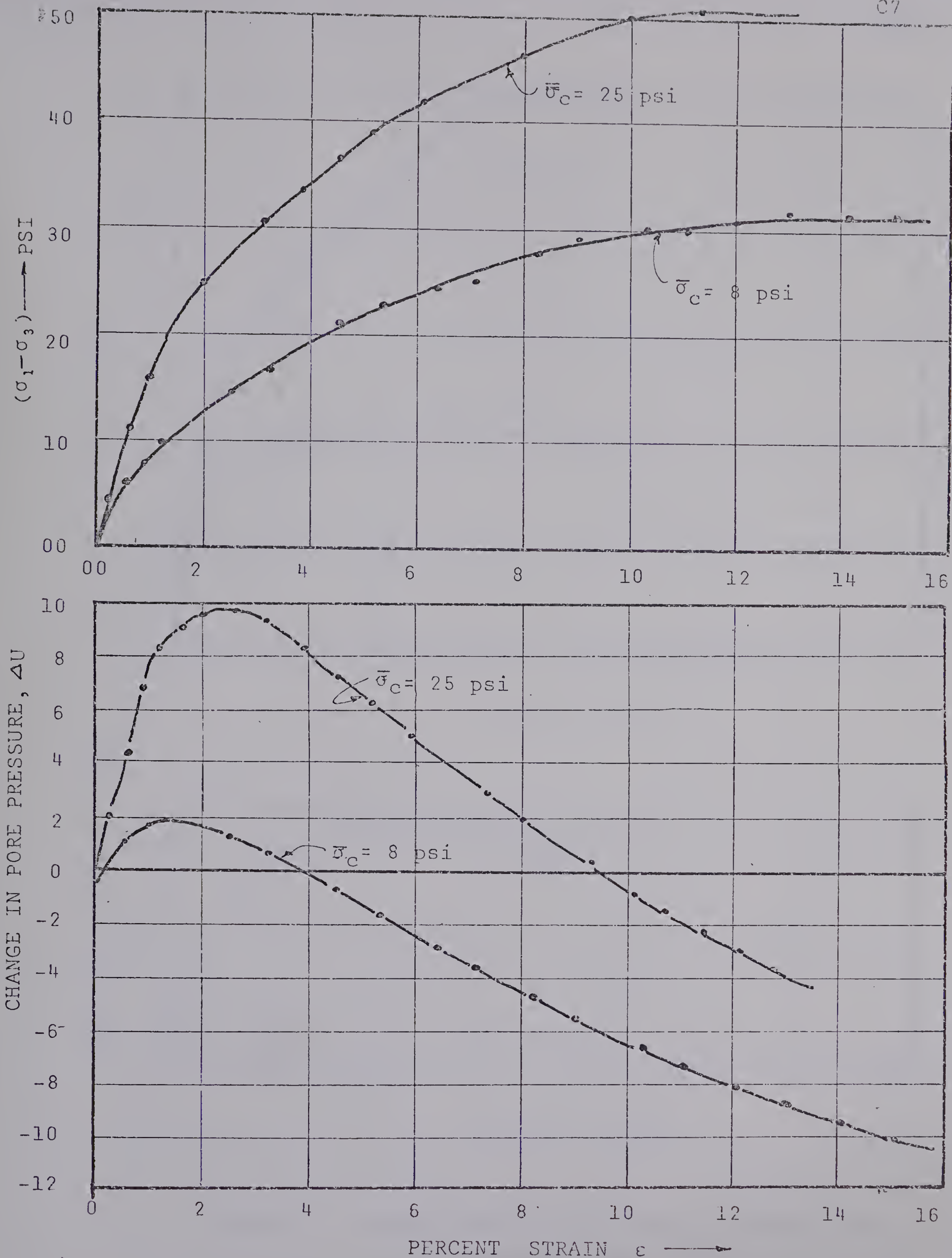
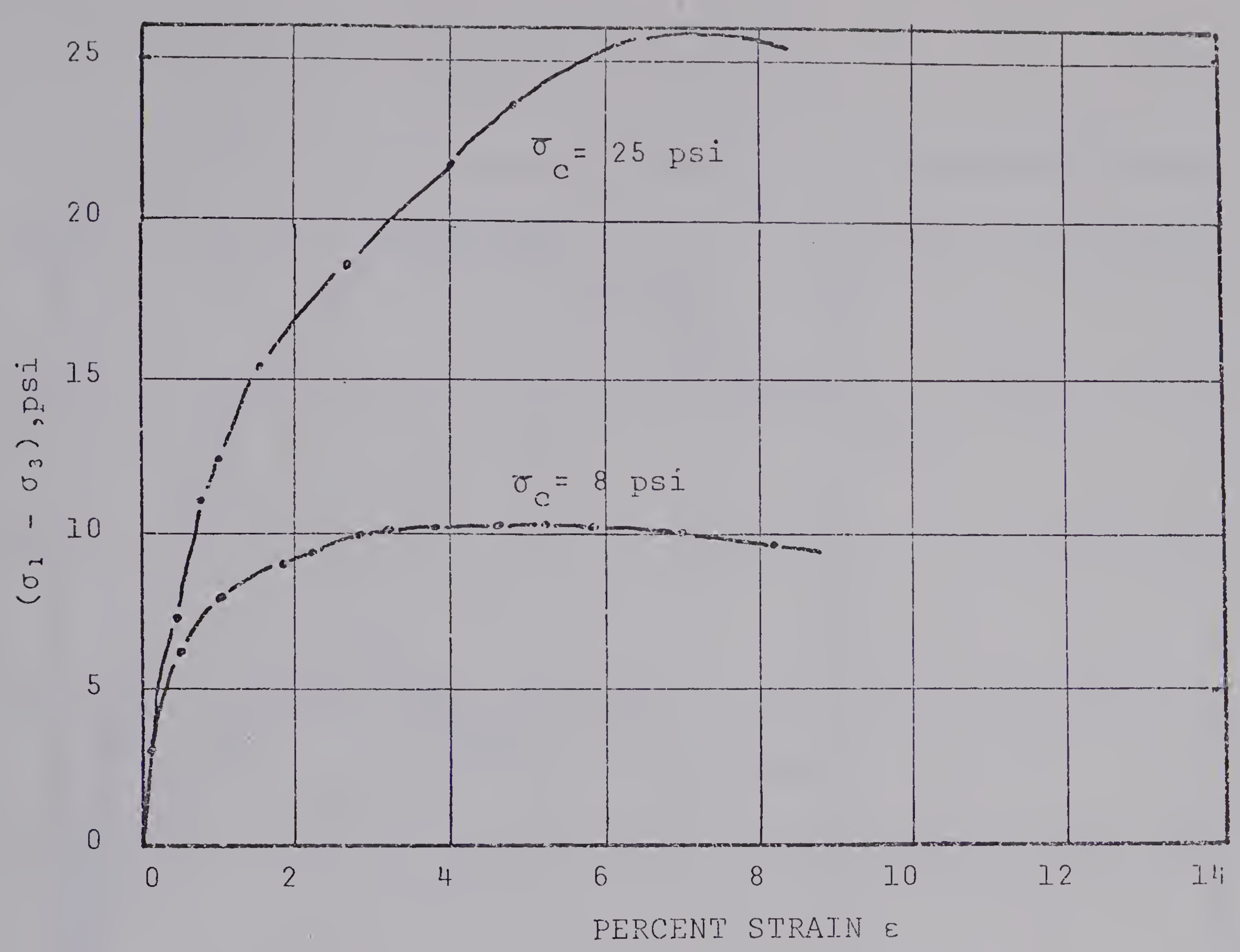



FIGURE C-6 STRESS-STRAIN RELATIONSHIP, SAMPLE ABL-3 / ABL-2
C-U UNDISTURBED, TEST SERIES 'B'



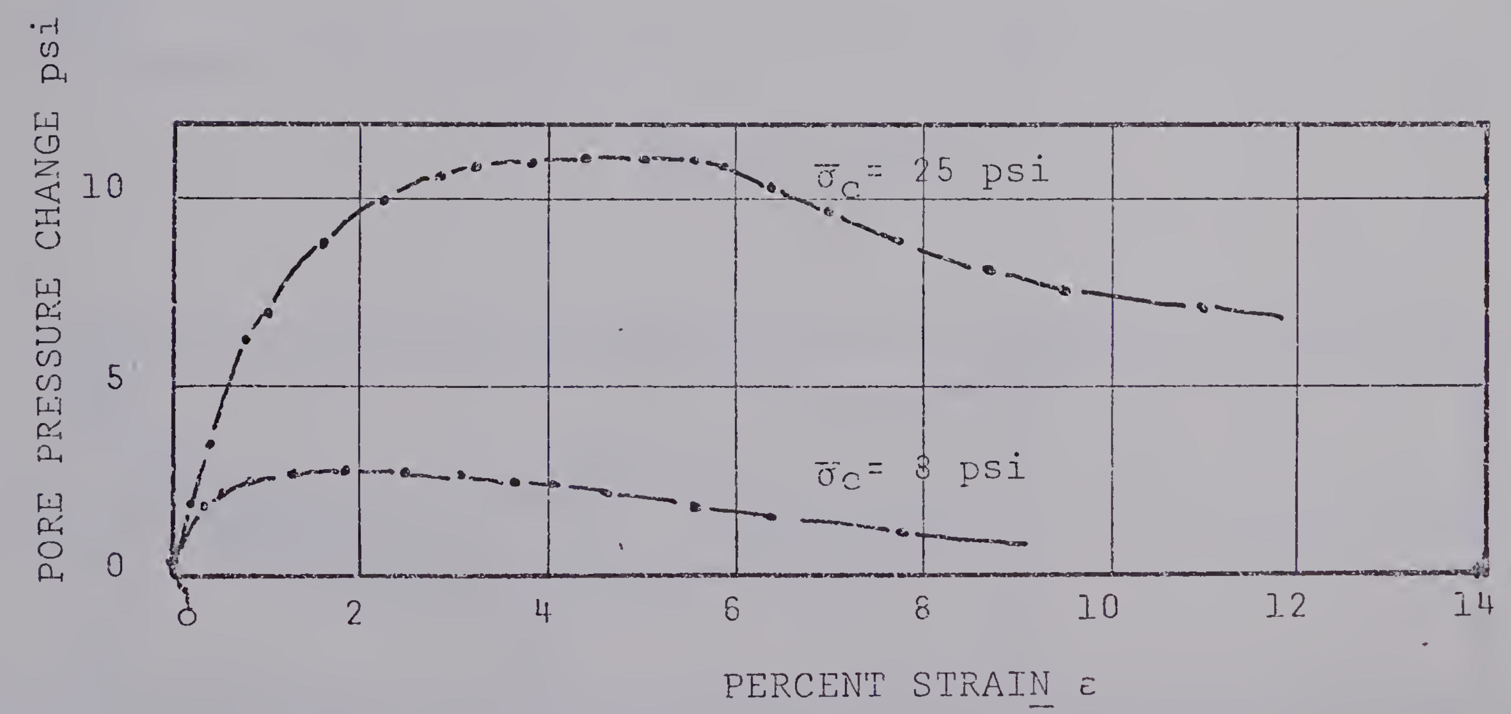
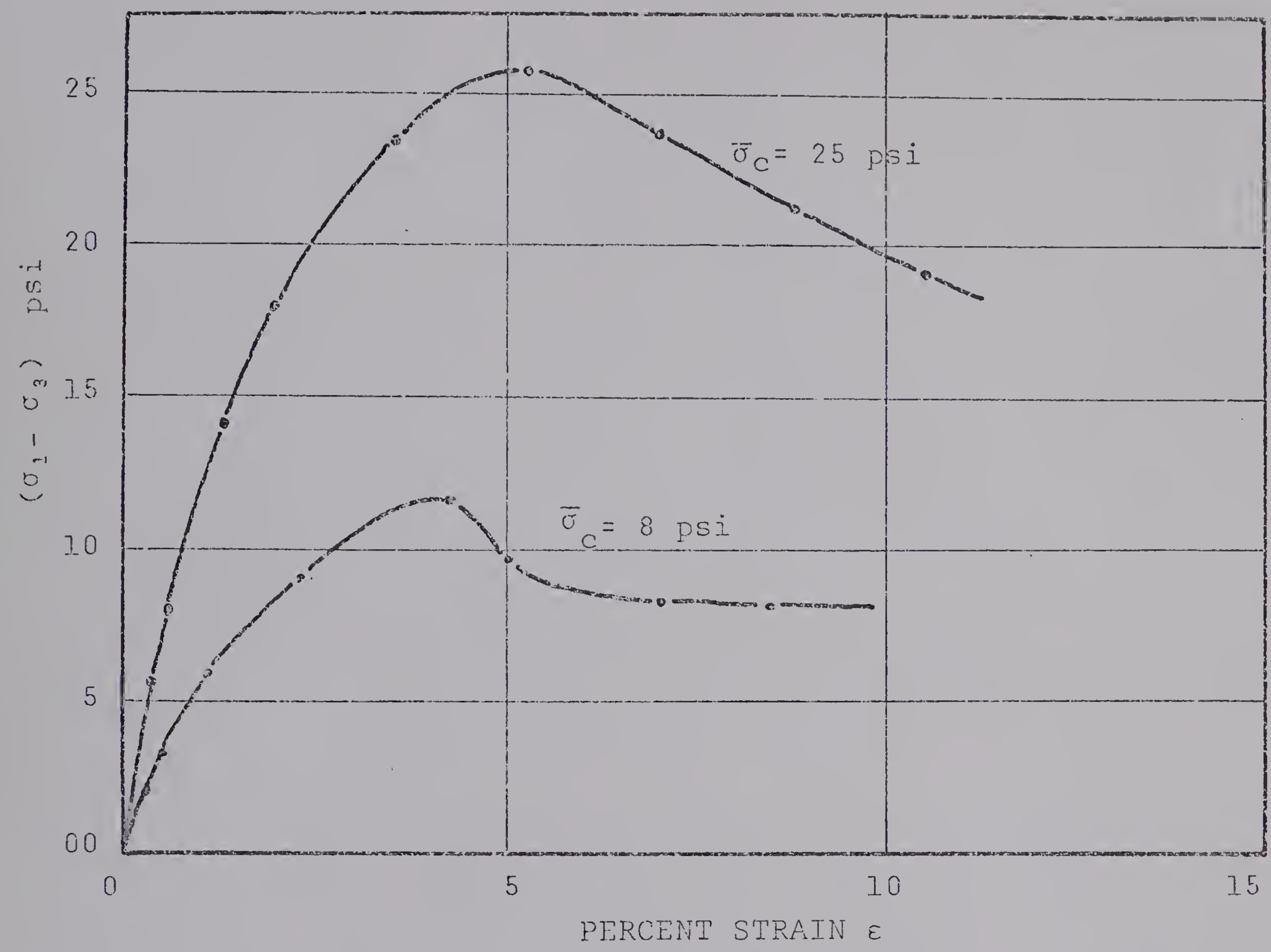




FIGURE C-7 STRESS-STRAIN RELATIONSHIP, SAMPLE ABL-2
C-U UNDISTURBED, TEST SERIES 'C'



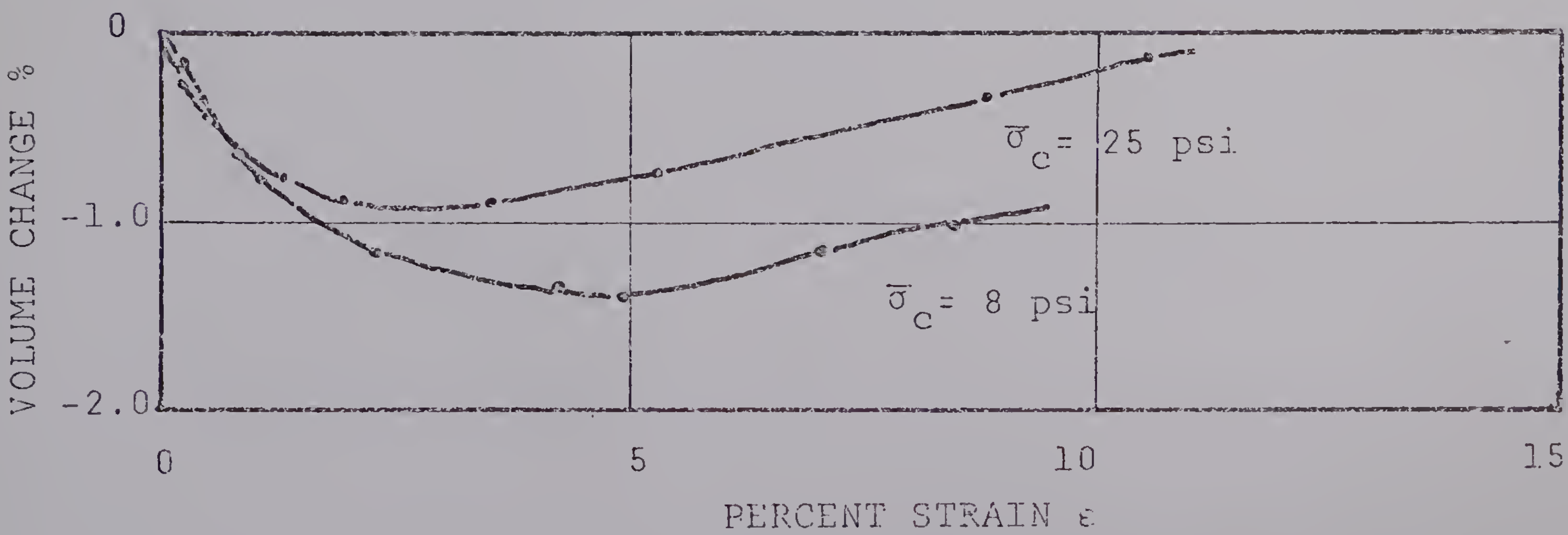




FIGURE C-8 STRESS-STRAIN RELATIONSHIP, SAMPLE ABL-2
C-D UNDISTURBED

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